EDITORS NOTES

General/Introduction

The practice of engineering geology in Colorado is as diverse and complex as Colorado's geology. Although no volume can completely encompass the breath of the history, current practice, and future trends, the papers in this publication have attempted to provide an overview of practice of engineering geology in Colorado – from the eastern plains to the peaks of the Rocky Mountains to the plateaus, mesas, and canyons of the western slope; from the perspective of private individuals, consultants, local, state, and federal government, and academia; and from notable case histories to current practice to future trends and research needs. The editors of this publication hope that you find this publication informative, enjoyable, and beneficial.

As is the case in any publication of this magnitude, there are unanticipated surprises that change the direction of the document. Regretfully, one of those disappointments was the loss of a number of quality papers summarizing some of the largest dam projects in Colorado. Due to security concerns following the events of September 11, 2001, it was not possible to publish these fine papers.

It is with great respect and admiration that we dedicate this publication to the memory of John B. Ivey. John served the profession of engineering geology in Colorado for nearly 50 years before his passing in July 2003. One of John's last contributions to the profession is his paper entitled "Engineering Geology for Relocation of a Highway in Glaciated Terrain, Climax Mine Area, Summit County, Colorado", co-authored with Jerome Hansen, included in the Transportation chapter of this publication.

Acknowledgements

This document would not have been possible without the efforts of many individuals who graciously donated their time, expertise, and energy to the successful production of this publication. The editors would like to thank the nearly 100 authors of the over 60 papers included in this document, for without them, this publication would not be possible. The editors would also like to thank the chapter editors and other reviewers for their efforts in recruiting authors, reviewing papers, coordination with authors, and providing comments on ways to improve the quality of the publication, including:

Rick Andrew
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In addition, the editors would like to offer special thanks to the following individuals:

Dawn Taylor Owens, Department of Natural Resources, for her review of each manuscript.
David Noe, Colorado Geological Survey, for his energy, enthusiasm, and guidance in making this publication a reality. We also appreciate his donation of the striking photograph on the front cover.
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Jim Wright and the Rocky Mountain Section, for making it financially possible to produce this document.
Kerry Cato, Special Publications Editor, Association of Engineering Geologists, for his assistance and direction during production of the publication.
Vince Reindl, Omnipress, for his helpful assistance and insights in producing this CD.

We acknowledge the inspiration of the other sections of the Association of Engineering Geologists who have produced similar publications and "set the bar" so high.

Disclaimer

The papers that comprise this volume have been edited for uniformity in organization, format, and overall clarity. They have also been reviewed for general content. The data, interpretations, and conclusions presented herein are solely those of the individual authors, however, and the findings and opinions expressed in these papers are not necessarily those of the Association of Engineering Geologists or the Colorado Geological Survey, Department of Natural Resources.

The Editors

Douglas D. Boyer, U.S. Bureau of Reclamation
Paul M. Santi, Colorado School of Mines
William Pat Rogers, emeritus, Colorado Geological Survey

DEDICATION

John B. Ivey
1927-2003

John B. Ivey graced not only his profession of engineering geology but also all those he came to know and touch. John will long be remembered for the two best reasons: his character and his adherence and contributions to his calling. John's secret to practice was the simple formula of openness, sincerity, diligence and honesty. We doubt that he knew otherwise and that his life ethic simply became his own canon of ethics in practice. John had a sincere interest in the well being of his brothers and sisters in practice and this came across in the most friendly of attitudes. In the words of John's stepson, Brian Webster, delivered at his memorial service, "Anyone who knew John knew that mediocrity was not his domain. He didn't do anything in which he didn't excel . . .. He didn't do anything halfway because his heart didn't "know" halfway. His heart overflowed with love, passion, sincerity, integrity and principle. This, in my mind, is what established John's greatness." No-nonsense problem solving was an Ivey specialty.

John was president of the Association of Engineering Geologists in 1980. Earlier he served as chairman of the Denver Section (1969), Chairman of the AEG Ethics and Practices Committee (1973-1978), and as Annual Meeting Chairman in 1974. AEG recognized these contributions broadly in citing John in 1987 as the third person honored by the Floyd T. Johnston Service award.

In addition to AEG, John held membership in the American Association of Petroleum Geologists, American Geophysical Union, American Institute of Professional Geologists (President of the Colorado Section, 1970), American Society of Photogrammetry, Geological Society of America (Fellow, 1980), International Association of Engineering Geologists, Rocky Mountain Association of Geologists and Society of Mining Engineers of AIME. John held professional registration as an Engineering Geologist in California and Oregon and as an Engineer in Colorado.

STATE OF COLORADO

EXECUTIVE CHAMBERS

136 State Capitol
Denver, Colorado 80203-1792
Phone (303) 866-2471

Bill Owens
Governor

Greetings:

Colorado's beautiful and rugged mountains, endless plains, and redrock plateaus all have their origins in geologic history. Our colorful history, from earliest human settlement through the legendary mining boom and expansion of the agriculture industry, to today's statewide growth patterns, has been influenced to a great degree by the state's geology.

In Colorado, geological processes sometimes present dire threats to the public. We appreciate the work performed by engineering geologists who deal with hazards posed by landslides, rockfalls, mudflows, avalanches, swelling and collapsing soil, evaporite karst, earthquakes, and radon. An understanding of geology has been a critical factor for other issues as well, including the siting and construction of dams, tunnels, pipelines, and transportation corridors, mapping and development of mineral resources, and maintaining water quality and supply.

I am pleased to introduce "Engineering Geology in Colorado – Contributions, Trends, and Case Histories." This special publication by the Rocky Mountain Section of the Association of Engineering Geologists and the Colorado Geological Survey of the Department of Natural Resources contains an incredible compilation of facts, knowledge and experience pertaining to the state's geology, geological processes and geological hazards. It contains numerous instances where this knowledge has been used to benefit the public health, safety and welfare.

This volume is essential reading for anyone doing geology-related work for civil projects in Colorado. It is an invaluable history about how we have dealt the state's scenic, yet complicated and sometimes problematic, geology.

Sincerely,

Bill Owens.

Bill Owens
Governor

ENGINEERING GEOLOGY IN COLORADO
Contributions, Trends, and Case Histories

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Chapter Editors Jerry Higgins, Bob Schuster, and Bill Smith

Faulting/Earthquake Hazards

Chapter Editors Vincent Matthews, III and Jon Ake

Land Use Planning

Chapter Editor Karen Berry

ENGINEERING GEOLOGY IN COLORADO
Contributions, Trends, and Case Histories

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Overview of the Geology of Colorado – Part I: Physiography, Climate, Geologic Setting, and Geologic History - *David C. Noe, Vincent Matthews, III, and Ralf Topper* (This paper is derived from two copyrighted publications from the Colorado Geological Survey – Special Publications 52 and 53 – used with permission; use of this material is subject to the CGS copyrights)

Overview of the Geology of Colorado – Part II: Mineral and Mineral Fuel Resources, Water Resources and Water Quality, and Geologic Hazards - *David C. Noe, Vincent Matthews, III, and Ralf Topper* (This paper is derived from two copyrighted publications from the Colorado Geological Survey – Special Publications 52 and 53 – used with permission; use of this material is subject to the CGS copyrights)

Difficult Geological Conditions on Steep Slopes That Required Innovative Slope Stability and Retaining Wall Systems for Highway Widening at Snowmass Canyon – *Benjamin P. Arndt, Richard Andrew, and Roger Pihl*

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OVERVIEW OF THE GEOLOGY OF COLORADO PART I: PHYSIOGRAPHY, CLIMATE, GEOLOGIC SETTING, AND GEOLOGIC HISTORY

David C. Noe; Vincent Matthews, III; and Ralf Topper
Colorado Geological Survey, 1313 Sherman Street, Room 715, Denver, CO 80203

Key Terms: general geology, physiography, climate, geologic history, Quaternary geology, geomorphic landforms

ABSTRACT

Colorado's mineral and water resources, and its geologic hazards, are closely associated with the state's geography, climate, surface and subsurface geology, geomorphology, and past and present geologic processes. In terms of its geography, Colorado has three major physiographic provinces that roughly trend north-south through the state – the Great Plains, Southern Rocky Mountains, and Colorado Plateau provinces. Two other provinces – The Middle Rocky Mountains and the Wyoming Basin – occupy the far northwest corner of the state. Colorado's location, far inland from any ocean, produces a semiarid climate with hot summers and cold winters. Its varied topography greatly influences patterns of precipitation, temperature, and air movement. Except for the high mountains, most of the state has a deficit water balance as a result of low precipitation and high evapotranspiration rates. Colorado contains an abundance of igneous, metamorphic, and sedimentary rock types, with rock formations representing every major geologic era and period, with the exception of the Silurian Period. The Cretaceous- to Tertiary-age structural deformation episodes associated with the Laramide uplift of the Rocky Mountains, and subsequent late Cenozoic volcanism, block faulting, and uplift have produced Colorado's varied and complex geology and physiography. The resulting landscape consists of structural basins filled with Paleozoic, Mesozoic, and Cenozoic sedimentary rock formations, separated by structural uplifts cored by Precambrian crystalline rocks. During the Quaternary Period, glaciers, water, wind, gravity, volcanic eruptions, and faulting modified the Tertiary landscape to produce the recent sediment deposits and landforms we see in Colorado today. Many of these geomorphic processes are active today. An understanding of the state's geology and geologic history – including the nature of its bedrock formations, Quaternary deposits, and geomorphic processes – is crucial in order to wisely and efficiently develop Colorado's mineral and water resources, and to protect the public from its many geologic hazards.

INTRODUCTION

Colorado is a land of spectacular beauty, much of which is defined by its geology. It contains rich but finite mineral deposits and water resources that have been, and will continue to be, objects of exploration, exploitation, and opportunity. These resources are key to the state's past, present, and future booms and busts. Colorado's geology presents many serious hazards and constraints to development, which must be dealt with as the state's population grows into areas with difficult terrain and active geologic processes. An understanding of geology is crucial in order to wisely and efficiently develop Colorado's mineral and water resources and to protect the public from geology-related threats to life, health, and safety.

The purpose of this two-part paper is to present an overview of the geology of Colorado as a framework for more-specific, engineering geology papers that follow in this volume. In the first paper (Part I), we will present a greatly simplified discussion about Colorado's physiographic, climatic, and geologic settings, geologic history, and modern landforms and geologic processes. This will be followed in the second paper (Part II) (Noe et al., 2003) by discussions about the occurrence and distribution of the state's mineral, mineral fuel, and surface and ground-water resources, water quality, and geologic hazards. In addition, in Part II we will discuss the association of these resources and hazards with Colorado's geography, surface and subsurface geology, geomorphology, and past and present geologic processes.

A basic understanding of Colorado's geologic setting and geologic history is important for Engineering Geologists who conduct research and investigations in the state. Knowledge of the lithostratigraphy, paleodepositonal environments, and structure of rock units in an area, in addition to the general engineering properties of those formations of interest, gives the geologist a powerful, predictive "sixth sense" about what to expect at a particular site.

Such knowledge enhances the interpretation of geologic maps and allows for the application of experience from other geologically similar areas. It is important in the consideration of bedrock conditions and hazards for a variety of investigations, including those for dam sites and highway and tunnel alignments. An understanding of the local bedrock geology and geologic history may enhance specific investigations of hydrogeology, mineral deposits, rockfall or debris-flow source areas, rock-rooted landslides, karst terrain, and paleoseismicity.

Further Reading

There are several excellent volumes on Colorado's geology for the inquisitive reader who requires more information about these topics. Many of these references were written for specific professional or general audiences (e.g., oil and gas, ground water, students, hobbyists) and, unfortunately, many of these classic compilations are badly out of date and out of print. Seven of the more-useful reference books and maps are listed below.

Two recent compilations by the Colorado Geological Survey (CGS), references 6 and 7, are the main sources for many of the figures and narrative descriptions used in this paper.

1) *Geologic Atlas of the Rocky Mountain Region* by the Rocky Mountain Association of Geologists (RMAG) (Mallory, 1972). This large-format atlas contains detailed descriptions of the physical, historical, and economic geology of Colorado and is richly illustrated with photographs and full-color cross-sections and maps. Out of print.

2) *Colorado Geology*, also by RMAG (Kent and Porter, 1980). We recommend this scholarly volume as the authoritative introduction to Colorado Geology. It contains papers on Colorado's structural and tectonic framework, geologic history, and resources. It also

contains a paper (Booy, 1980) that describes several geologic hazards and the use of engineering geology as related to water supply, tunneling, and planning. Out of print.

3) *Prairie Peak and Plateau* (Chronic and Chronic, 1972). This very readable guidebook is aimed at an "educated non-technical" audience and covers the topics of physical, historical, and economic geology of Colorado. Out of print (but see item 7).

4) *Roadside Geology of Colorado* (Chronic and Williams, 2002). Also written for an "educated non-technical" audience, this second-edition guidebook describes the geology of Colorado along its highways, and is illustrated with photographs and maps.

5) *Geologic Map of Colorado* (Tweto, 1979). The definitive map of Colorado's geology at a statewide scale of 1:500,000. Recently, the U.S. Geological Survey has released a digital, GIS compatible version of this map (Green, 1992).

6) *Ground-Water Atlas of Colorado* (Topper et al., 2003). This newly released atlas shows the interrelationship between Colorado's geology, climate, surface water, ground water, and aquifer systems. It is richly illustrated with color photographs and maps.

7) *Messages in Stone: Colorado's Colorful Geology* (Matthews et al., in prep.). This profusely illustrated text is in final review and was written as a successor to *Prairie Peak and Plateau* (item 3). It is intended for a wide variety of audiences and serves as both an educational tool and a scenic guidebook, and will probably be published in 2004.

In addition to these larger compilations, there are a number of excellent technical papers that describe statewide or regional aspects of Colorado's geology (examples include, Tweto, 1964; Hansen and Crosby, 1982; Weimer, 1996). A full listing of such references is beyond the scope of this paper.

PHYSIOGRAPHY

Colorado is the eighth-largest state in the nation, with an area of over 104,000 mi^2 (269,000 km^2), and is unique in its varied landscapes and topography. It is America's highest state, with an average elevation of 6,800 ft (2,073 m) above sea level. Fifty-eight of Colorado's peaks soar to more than 14,000 ft (4,267 m), more than in all the other states combined, and more than 740 peaks exceed 13,000 ft (3,962 m). A number of high mountain ranges, covered with montane and subalpine forests, alpine tundra, and raw, glacier-gouged cliffs occupy the central portion of the state. However, much of Colorado, especially in the areas around its population centers, is comprised of flat or gently rolling topography on which grasslands predominate. Still, other areas of the state have an Old-West look, with scrubby, desert-like vegetation and rocky, steep-sided mesas and canyons.

This geographic variation is expressed in three major physiographic provinces that roughly trend north-south through the state – the Great Plains, Southern Rocky Mountains, and Colorado Plateau provinces (Figure 1). Portions of two other physiographic provinces – The Middle

Rocky Mountains and the Wyoming Basin – occupy the far northwest corner of the state. All of these provinces extend beyond Colorado and define regions in which structures, climate, relief, landforms, and geomorphic history are similar.

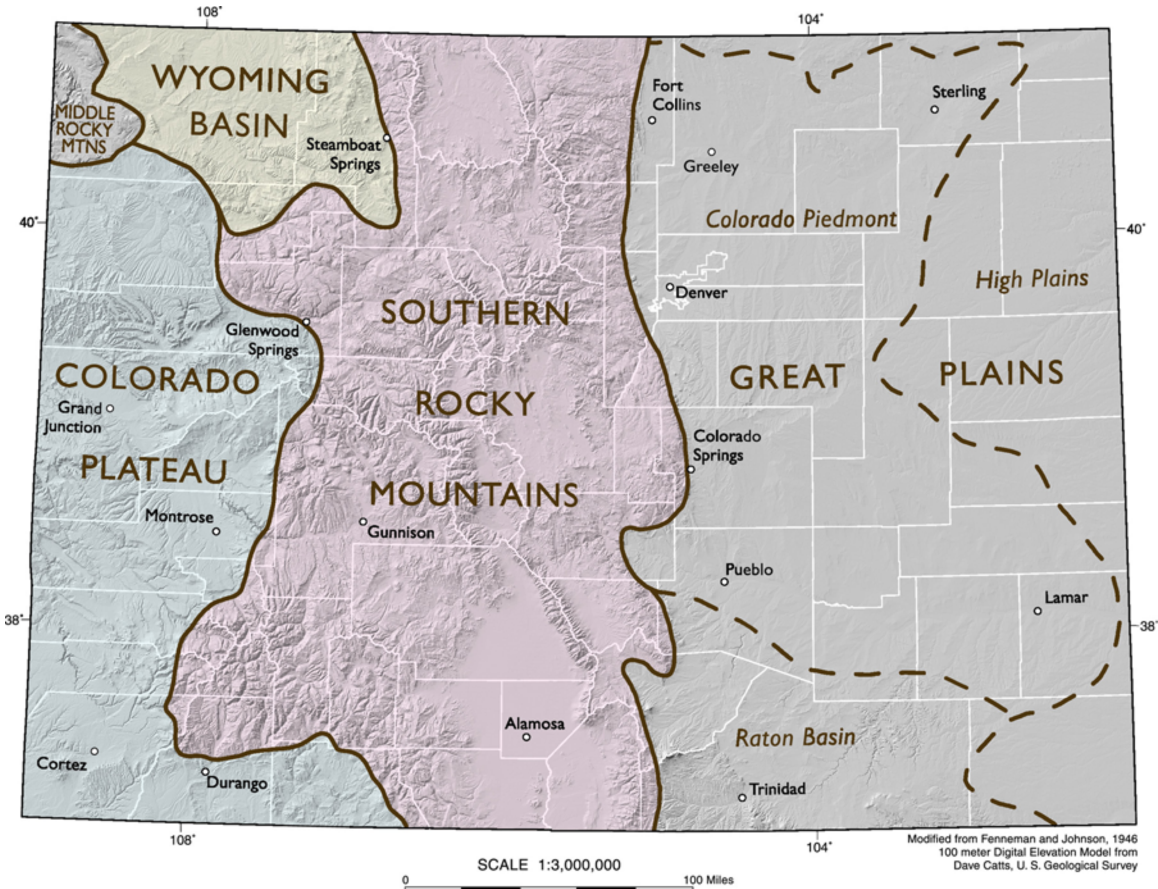

Figure 1. Colorado's physiographic provinces. From Topper et al. (2003); modified from Fenneman and Johnson (1946); 100-meter DEM data from David Catts, U.S. Geological Survey.

The Great Plains province encompass approximately 40 percent of the state, with the remaining area divided roughly equally between the plateau and mountainous regions. The main features of the Great Plains are large, flat drainage divides of rolling grassland that lie between and adjacent to the valleys of the South Platte and the Arkansas Rivers. These major rivers, which arise in the central mountains, gradually fall approximately 3,000 ft (914 m) in elevation as they cross the plains eastward from the foothills to the eastern border of the state. Colorado's lowest elevation is 3,350 ft (1,021 m) where the Arkansas River leaves the state. The province is subdivided into the High Plains, Colorado Piedmont, and Raton Basin sub-provinces.

The Southern Rocky Mountain province encompasses the center of the state and runs its entire north-south length. It is characterized by several distinct mountain ranges with elevations ranging from 6,000 ft (1,829 m) to over 14,000 ft (4,267 m). Colorado's highest point, Mt. Elbert, at 14,433 ft (4,399 m), is located in the central part of the state near Leadville. Valleys and high, intermontane parks such as North, Middle, and South Parks and the San Luis Valley separate the individual mountain ranges. The mountain ranges are heavily forested, except above timberline where tundra vegetation and rocky ground predominates. Grasslands predominate in the intermontane parks and valleys. The Continental Divide, formed by the crests of several mountain ranges, separates river basins draining east into the Gulf of Mexico from those that drain west into the Gulf of California. The headwaters of four of the West's major river systems (the North and South Platte, Arkansas, Rio Grande, and Colorado Rivers) are within this region (Figure 2).

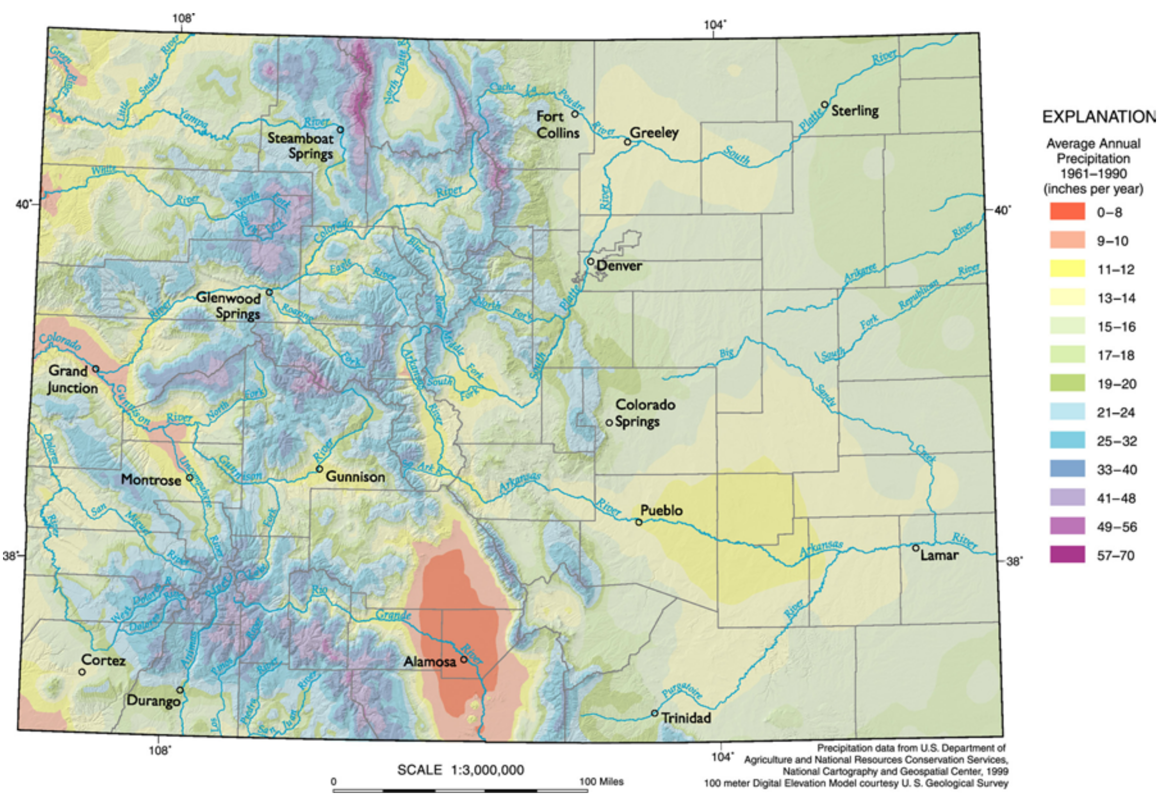

Figure 2. Average annual precipitation and major river systems in Colorado. From Topper and others (2003); modified from U.S. Department of Agriculture and Natural Resources Conservation Service (1999).

In western Colorado, the Colorado Plateau province consists of a series of high, relatively horizontally stratified plateaus and mesas that have been dissected by rugged canyons and deep, broad valleys associated with the major river systems. Some of the more pronounced upland areas include the Book Cliffs and Roan Cliffs, Battlement Mesa, and Grand Mesa near Grand Junction and Rifle, Uncompahgre Plateau near Montrose, and Mesa Verde near the southwestern corner of the state. The main rivers that cross this province include the San Juan, Animas, Dolores, Uncompahgre, Gunnison, Colorado, and White Rivers. Elevations within the province range between 5,000 and 10,000 ft (1,524 and 3,048 m).

The Middle Rocky Mountain province consists of 7,000 to 9,000 ft high (2,134 to 2,743 m), forested plateaus that are an eastward extension of Utah's Uinta Mountains. These plateaus are deeply dissected by the Green and Yampa Rivers.

The Wyoming Basin province is marked by sparsely vegetated, low-relief badlands that are drained primarily by the Little Snake River. Elevations in the Wyoming Basin portion of Colorado range between 6,000 and 7,500 ft (1,829 and 2,286 m).

CLIMATE

Colorado's location, far inland from any ocean, produces a semiarid climate with hot summers and cold winters. Its varied topography greatly influences patterns of precipitation, temperature, and air movement. Whereas air can move freely over the eastern plains, the mountains can act as barriers to air movement. The intermontane valleys and parks are subject to extremely cold winter temperatures because of restricted air movement.

Precipitation varies tremendously in Colorado (Figure 2). The average annual precipitation, statewide, is approximately 17 in (43 cm) (Colorado State University, 2002). Colorado's varied topography results in regions such as the San Luis Valley receiving an annual average of less than 12 in (30.5 cm) of precipitation, while the adjacent San Juan Mountains receive in excess of 40 inches (102 cm). The variability in annual precipitation from year-to-year throughout the state, along with a history of periodic droughts, is a cause of great concern for water managers and the general population.

The Great Plains and Colorado Plateau physiographic provinces generally have abundant sunshine, low relative humidity, large daily temperature variations, high to moderate winds, and little precipitation. Average annual precipitation in the eastern plains of Colorado ranges from 12 to 16 in (30.5 to 40.6 cm). The dissected and varied topography of the Colorado Plateau produces varied micro-climatic conditions. Valleys and basins between mesas may exhibit semi-arid, desert-like conditions, while alpine climatic conditions can exist at the higher altitudes. At elevations below 9,000 ft (2,743 m), average annual precipitation ranges from about 8 to 18 in (20 to 46 cm), while the mesa and mountain ranges receive in excess of 32 in (81 cm). Winter and spring storms produce most of the precipitation in this region. Summer thunderstorms, although brief, can often be very intense, producing 20 to 40 percent of the annual precipitation (Robson and Banta, 1995).

Precipitation within the Southern Rocky Mountain physiographic province is more consistent on an annual basis, though highly variable by elevation. A majority of the precipitation falls as snow from Pacific winter storms tracking eastward across the mountain ranges. This air movement tends to produce more precipitation on the windward (western) side of the ranges than on the leeward (eastern) side. Average annual snowfall ranges from 6 ft (1.8 m) to more than 35 ft (10.7 m) (Robson and Banta, 1995). Average annual precipitation in the mountain ranges of Colorado ranges from 30 in (76 cm) to over 60 in (152 cm).

In Colorado's semi-arid climate, most of the precipitation that falls on the land surface is lost through evapotranspiration (water loss under the combined effects of evaporation and plant transpiration). Abundant sunshine, clear skies, low relative humidity, wind, and moderate temperatures result in large rates of evaporation over much of the state. The annual rate of potential evaporation, as measured by standard weather bureau Class A evaporation pans, ranges from about 45 in (114 cm) in the west-central portion of the state to 85 in (216 cm) in the extreme southeast corner (Farnsworth et al., 1982). Statewide averages show that approximately 81 percent of Colorado's precipitation returns to the atmosphere through evapotranspiration (Litke and Evans, 1987). In the foothills west of Denver, measured evapotranspiration rates have varied annually from 75 to 97 percent of precipitation (U.S. Geological Survey, 2001).

Water Budget

The average annual water balance in Colorado, calculated by subtracting the average annual potential evapotranspiration from the average annual precipitation, is shown in Figure 3. This map shows that the mean annual water balance is a deficit over most of the state, with only the higher mountain regions producing surplus water. In general, in areas where evapotranspiration rates are in excess of precipitation, most surface water and soil moisture will be removed into the atmosphere before the water can infiltrate into the subsurface. However, potential and actual evapotranspiration may vary markedly, depending in part on how the precipitation is distributed through time, both seasonally and episodically.

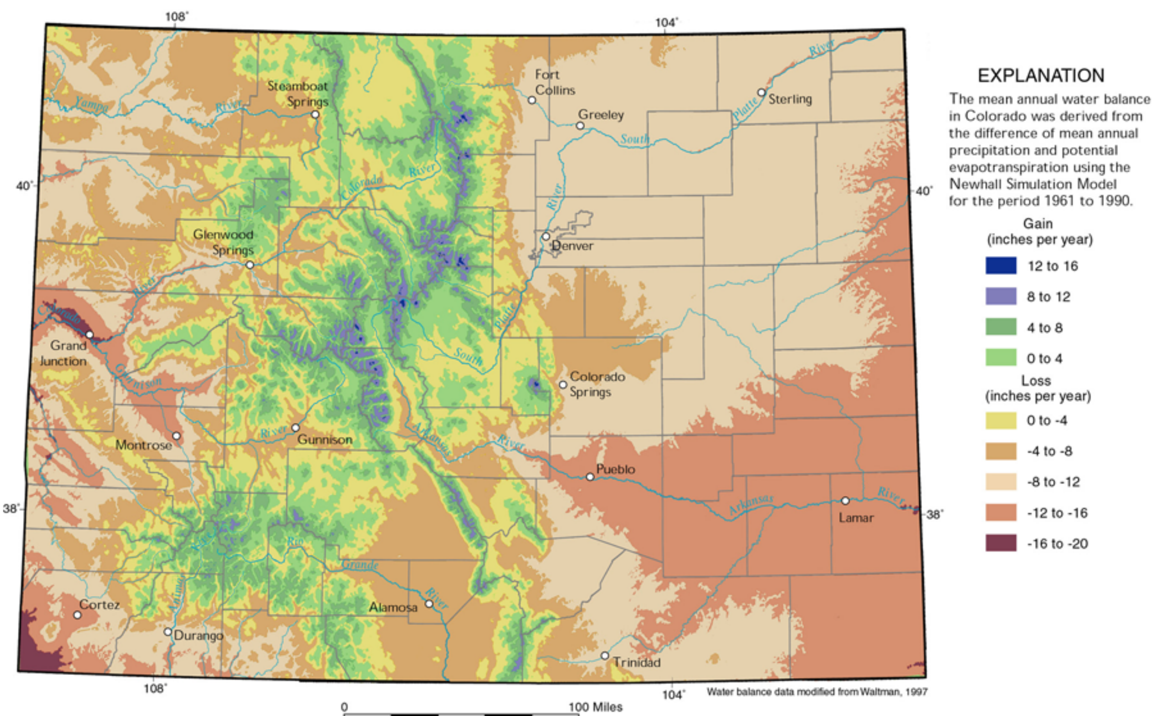

Figure 3. Average annual water balance in Colorado. From Topper and others (2003); modified from Waltman (1997).

This deficit water balance is an important factor that affects Colorado's surface-water supplies and recharge to ground-water aquifers. It is also an important factor with regard to engineering geology, as most of Colorado's soils are in a perpetually dry state (with the obvious exception of the higher-elevation mountains and other localized areas). Episodic, natural events that increase the soil moisture, such as seasonal precipitation or large precipitation events, and longer-term, human-induced activities such as drainage rerouting, septic-system return flows, and lawn or crop irrigation, are critical engineering geologic considerations.

GEOLOGIC SETTING

Colorado contains an abundance of igneous, metamorphic, and sedimentary rocks, which span billions of years of Earth's history and represent a variety of depositional environments. The state's varied and complex geology and physiography have been produced by deformation associated with the two most-recent uplifts of the Rocky Mountains (the Laramide uplift episode during Cretaceous to early Cenozoic time, and the later uplift episode during late Cenozoic time). As a result, Colorado is divided into a series of structural uplifts and basins (Figure 4). The Great Plains, Colorado Plateau, and Wyoming Basin provinces generally contain thick sequences of flat-lying to locally folded sedimentary rocks within areally extensive structural basins. The Southern and Middle Rocky Mountain provinces are comprised of a complex assortment of igneous (both intrusive and extrusive), metamorphic, and sedimentary rocks within numerous, variably sized structural uplifts and basins.

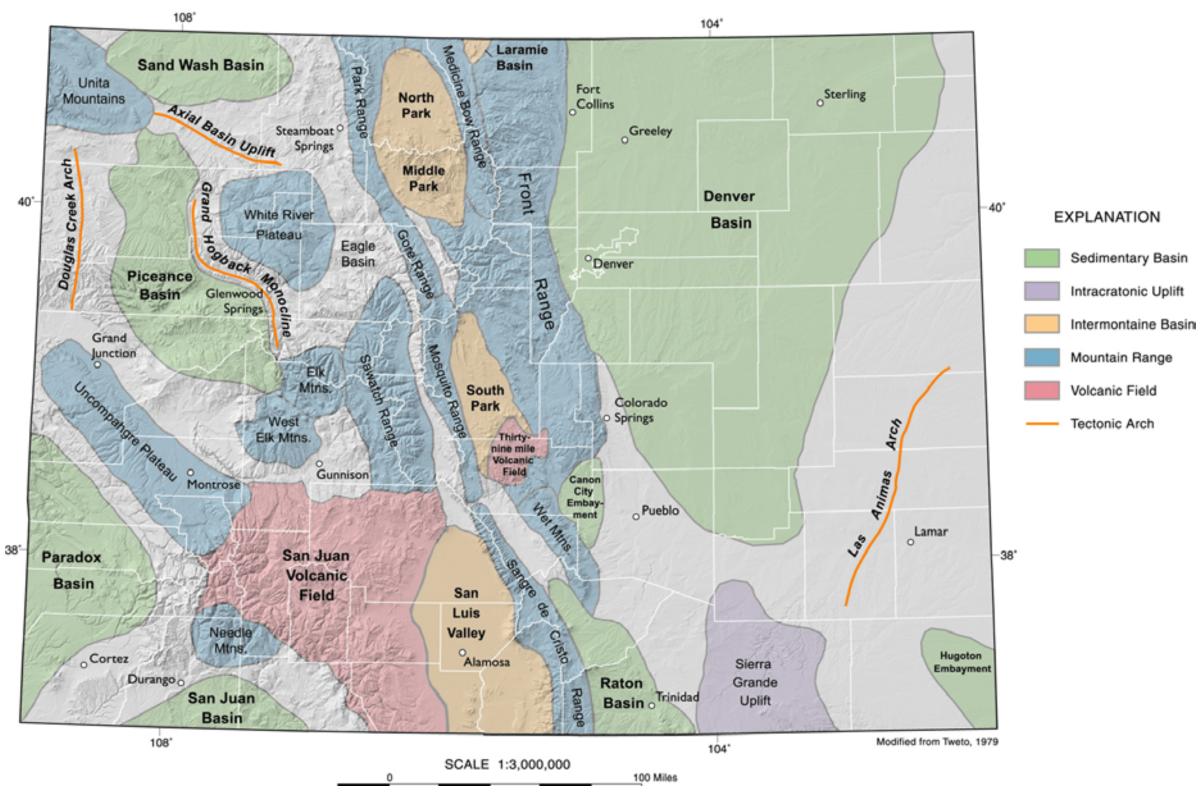

Figure 4. Major tectonic and structural features in Colorado. Modified from Tweto (1979).

Colorado contains rock formations from every major geologic era and period (with the exception of the Silurian Period, of which only a few remnants have been found). The surficial distribution of Colorado's major geologic units is presented in a simplified geologic map (Figure 5). A geologic cross-section (cross-section A-A', Figure 6), taken from the west-central part of the state from the Utah line to the eastern plains, shows a succession of structural basins filled with Paleozoic, Mesozoic, and Cenozoic sedimentary rock formations, separated by structural uplifts cored by Precambrian crystalline rocks. Each of the major structural basins has a somewhat unique assemblage of geologic formations, and in some instances the stratigraphic nomenclature varies from place to place. A stratigraphic nomenclature chart, by basin, of the state's Paleozoic, Mesozoic, and Cenozoic formations is included as Appendix A.

GEOLOGIC HISTORY

The geologic story deciphered from the rocks of Colorado recounts multiple structural events that raised mountain ranges, each to be subsequently eroded and partially buried in their own debris. Some of these rocks present evidence of ancient, shallow seas with their associated beaches, deltas, and swamps sweeping across the land, and of deserts undulating with dune fields. In more recent geologic time, the rocks record a history of large, active volcanic fields that covered the landscape with flowing lava and filled the air with volcanic ash. Over central Colorado, the landscape resulting from this geologic history is modified by the work of glacial ice that sculpted mountain peaks and scoured valleys, leaving thick layers of accumulated sediments across the land as glaciers retreated and melted. Beyond the glacier limits, extensive alluvial outwash and eolian sands and silts were deposited.

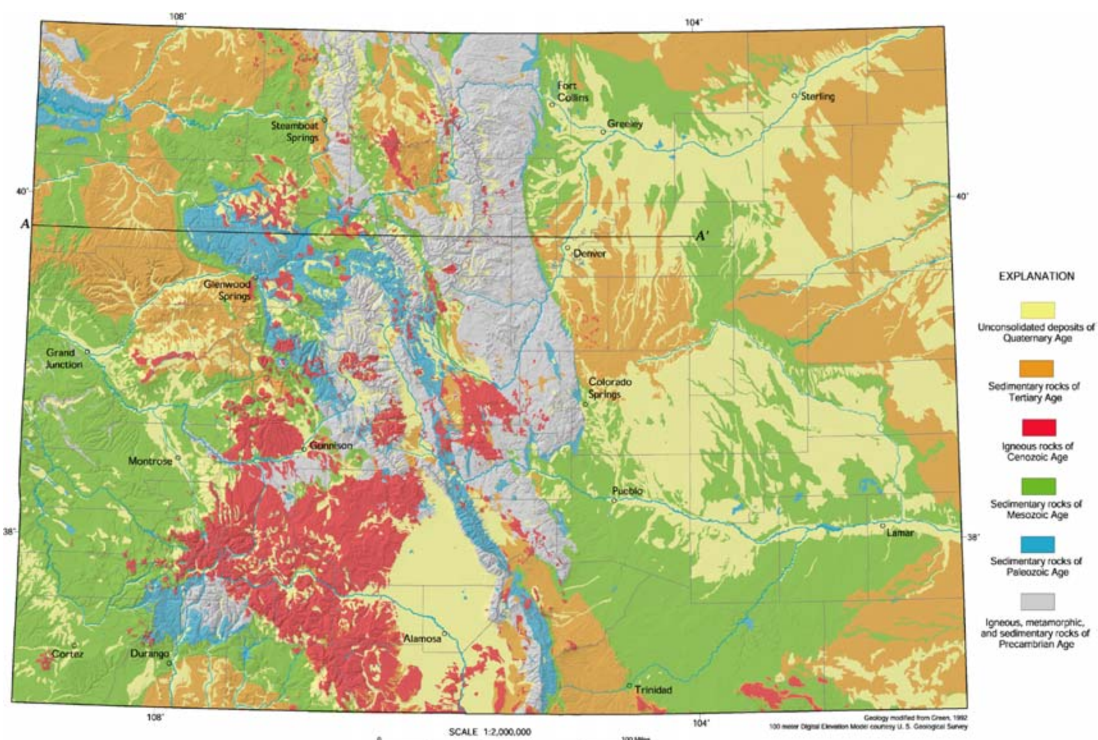

Figure 5. General geologic map of Colorado, by geologic era and period. From Topper and others (2003); modified from Tweto (1979) and Green (1992).

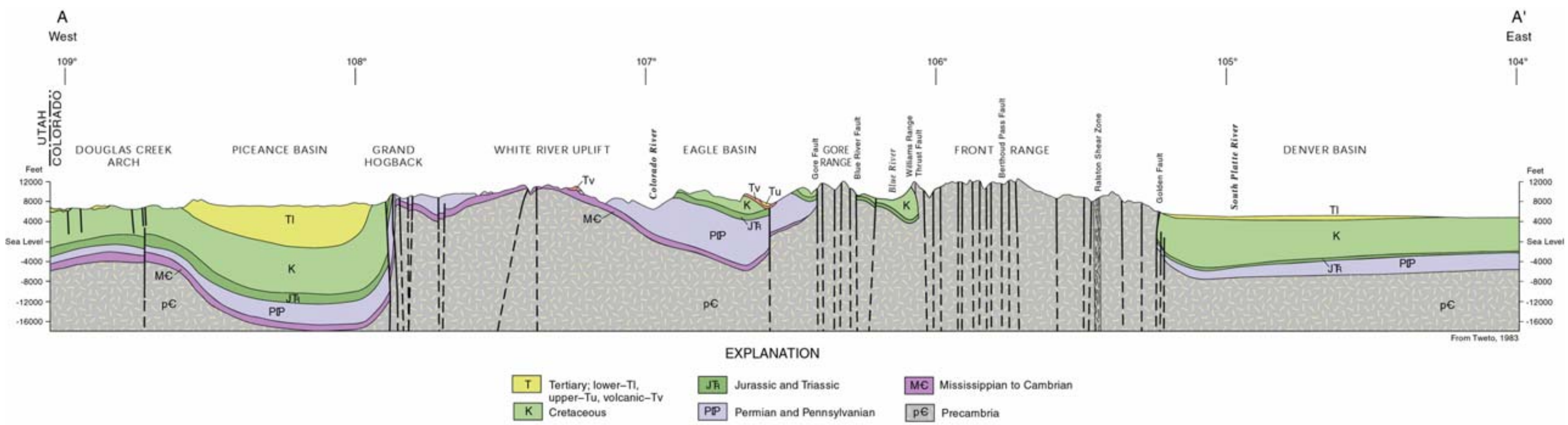

Figure 6. General geologic cross-section A-A' across west-central Colorado. See Figure 5 for location. From Topper and others (2003); modified from Tweto (1983).

The following section contains a brief geologic history of Colorado, along with maps showing the generalized surficial (outcrop) distribution of geologic units from a particular period. The names of pertinent rock formations, primarily those in the central part of Colorado, are included in these descriptions. For the Quaternary Period, the history shifts to a discussion of modern depositional environments and deposits. A summary of the major geologic events occurring within Colorado during each period of geologic time is depicted in Figure 7.

Era	Period	(Millions of years ago)	Major Geologic Events in Colorado
CENOZOIC	Quaternary	Present- 1.8	Development of present topography, "Ice Ages," Huge dune fields, Widespread mammalian extinction, Widespread faulting, Basaltic volcanoes, Development of caves.
	Tertiary	1.8-65	Rio Grande rifting and regional uplift by block faulting (to present elevations); Major canyon cutting; Catastrophic volcanoes erupting; Erosion and basins developing; Laramide mountain building; Igneous plutons intruding and mineralization along the Colorado Mineral Belt.
	K/T Boundary	65	Asteroid impact causes worldwide extinction of plants and animals. End of the "Age of Dinosaurs," beginning of "Age of Mammals."
MESOZOIC	Cretaceous	65-144	Subtropical to tropical climate; Shifting shorelines of Western Interior Seaway; Deep- and shallow-marine and non-marine conditions. Onset of Laramide mountain building; Final retreat of marine waters. Dinosaurs.
	Jurassic	144-206	Lakes, swamps, braided and meandering streams; Repeated invasion and withdrawal of sea before final withdrawal; Coastal dune deposits, Dinosaurs.
	Triassic	206-251	Semi-arid and arid conditions; Mudflats, alluvial plains, dune fields adjacent to eroding Ancestral Rockies; Deposition of "redbeds;" Dinosaurs dominate life forms.
PALEOZOIC	Permian	251-290	Dune fields; Continued erosion of Ancestral Rockies; Deposition of coarse "redbeds;" Mass extinction.
	Pennsylvanian	290-323	Widespread karst development on Mississippian limestones; Shallow seas and evaporite basins; Rise of the Ancestral Rockies; Erosion of great volumes of rock and deposition into alluvial fans
	Mississippian	323-354	Widespread shallow sea.
	Devonian	354-417	Widespread shallow sea; Intrusion of kimberlite diamond pipes.
	Silurian	417-443	Widespread shallow sea; Uplift and erosion.
	Ordovician	443-490	Local deepening of seas and cyclic sea-level rise and fall. First vertebrates in the world.
	Cambrian	490-545	Invasion of sea from west and east; Dike intrusion; Rapid development of hard-bodied life forms.
PRECAMBRIAN PROTEROZOIC EON		545-2,500	A major continent/island-arc collision; Three periods of regional metamorphism; Three periods of folding; Formation of major shear zones; Three periods of granitic intrusions; A period of mafic dike intrusion; Deposition of a thick sequence of continental sedimentary rocks.
PRECAMBRIAN ARCHAEN EON		2,500-4,500?	Rifting, granitic intrusion, regional metamorphism and folding. Metamorphism at 2.7 billion years ago.

Figure 7. Geologic time scale and summary of major geologic events occurring within Colorado. Compiled by the Colorado Geological Survey.

The discussions mention numerous, geographic-location names that are not shown on the outcrop-distribution maps that follow; interested readers who wish to find these various locations should equip themselves with a highway map of Colorado or a larger-scale geologic map such as the one by Tweto (1979).

The Precambrian Era

The Precambrian era spans the period from the origin of the Earth, estimated to be approximately 4.6 billion years old, to approximately 543 million years ago. The oldest known Precambrian rocks in Colorado are about 2.7 billion years old and are represented by meta-sediments exposed in the Uinta Mountains in the very northwest corner of the state. The majority of Precambrian-aged rock exposures in Colorado are found within the central Rocky Mountain region (Figure 8). These outcrops are primarily composed of metamorphic gneiss and igneous granite.

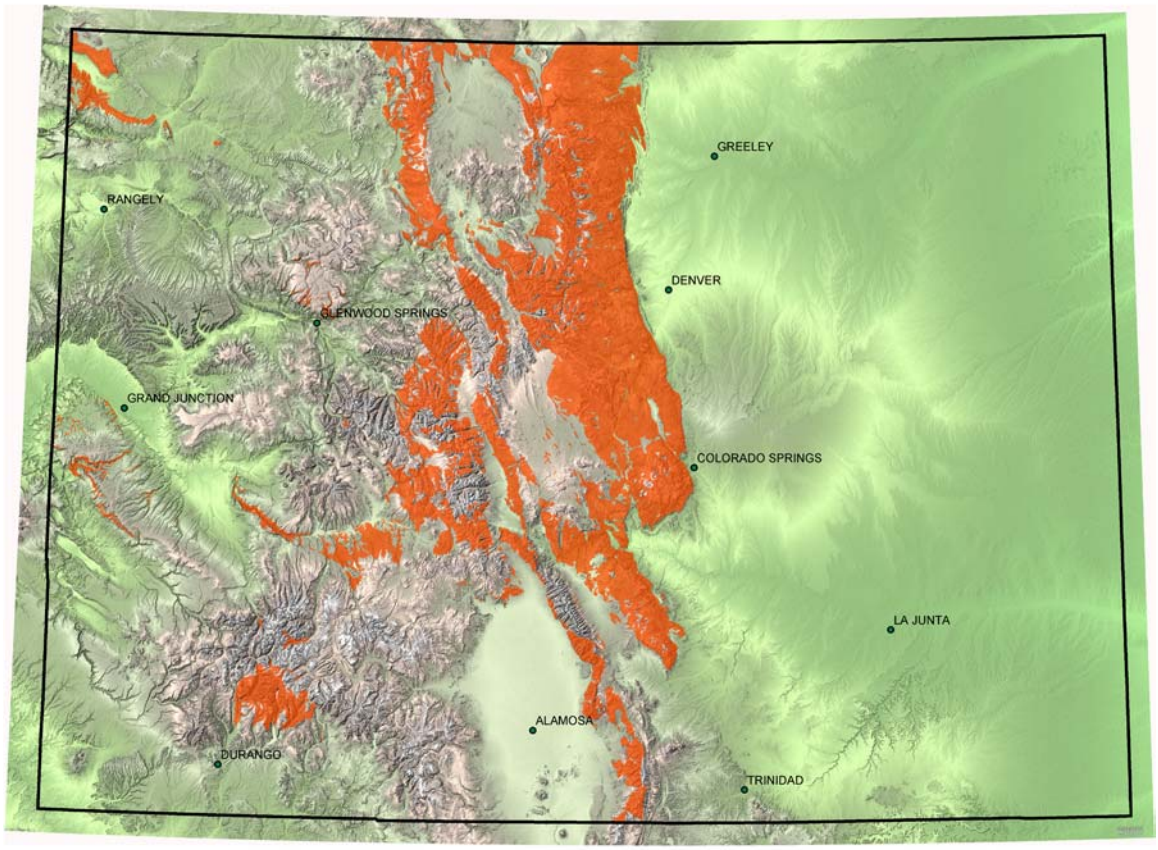

Figure 8. Distribution of Precambrian rock outcrops in Colorado. From Matthews and others (in prep.); modified from Tweto (1979).

Details of the ages and origins of the state's metamorphic rocks are uncertain, although there is evidence of mountain-building episodes, volcanism, and subsequent erosion and deposition of thick sequences of sediments during Precambrian time. These sediments have been subsequently transformed into today's metamorphic rocks by burial, pressure, and heat. The Black Canyon of the Gunnison River contains spectacular exposures of gneiss and schist with cross-cutting dikes of pegmatite (Figure 9). Other examples of metamorphic rocks in Colorado include gneiss from the Front Range, the Wet Mountains near Canon City, the Uncompahgre Plateau near Gateway, and the Needle Mountains near Durango. Quartzite is exposed in the Big Thompson and Coal Creek canyons in the Front Range. Precambrian-age sedimentary to meta-sedimentary rocks, primarily sandstone-quartzite and shale-argillite, are exposed in the area surrounding Browns Park in the northwest corner of the state.

There are several, large granitic batholiths of Precambrian age that cover large areas of the central and northern mountains (Figure 10). The most notable is the Pikes Peak batholith, which intruded the Pikes Peak Granite (1.0 billion years) to the west of Colorado Springs. Other batholiths intruded the Silver Plume Granite (~1.4 billion years) and the Boulder Creek Granite

Figure 9. Black Canyon of the Gunnison River, near Montrose. Photo by Jason Wilson, Colorado Geological Survey.

(~1.7 billion years) along the Front Range; these granites have age-equivalent counterparts scattered across other mountain ranges of the state.

The Paleozoic Era

The Paleozoic Era, comprising seven geologic periods, spans the time period from 543 to 248 million years ago. It began with an explosive expansion of life and ended with a dramatic mass extinction of marine invertebrates. In Colorado, widespread shallow seas dominated the early portion of the Paleozoic era. This relatively stable, cratonic environment, coupled with the activity of offshore currents and marine organisms, resulted in the deposition of sandstone, limestone, and dolomite. Uplift of the land and subsequent receding of the seas in the late Paleozoic (Pennsylvanian time) created the Ancestral Rocky Mountains. Their subsequent erosion produced thick deposits of coarse-grained red beds. Isolated shallow seas produced restricted evaporite basins, while shales and sands were deposited in other parts of the state.

Colorado spent the early two-thirds of the Paleozoic in the southern hemisphere (see Blakey, 2003, and Scotese, 2003 for illustrated, paleogeographic reconstructions of the position of the North American continent relative to the equator through geologic time). By Pennsylvanian time

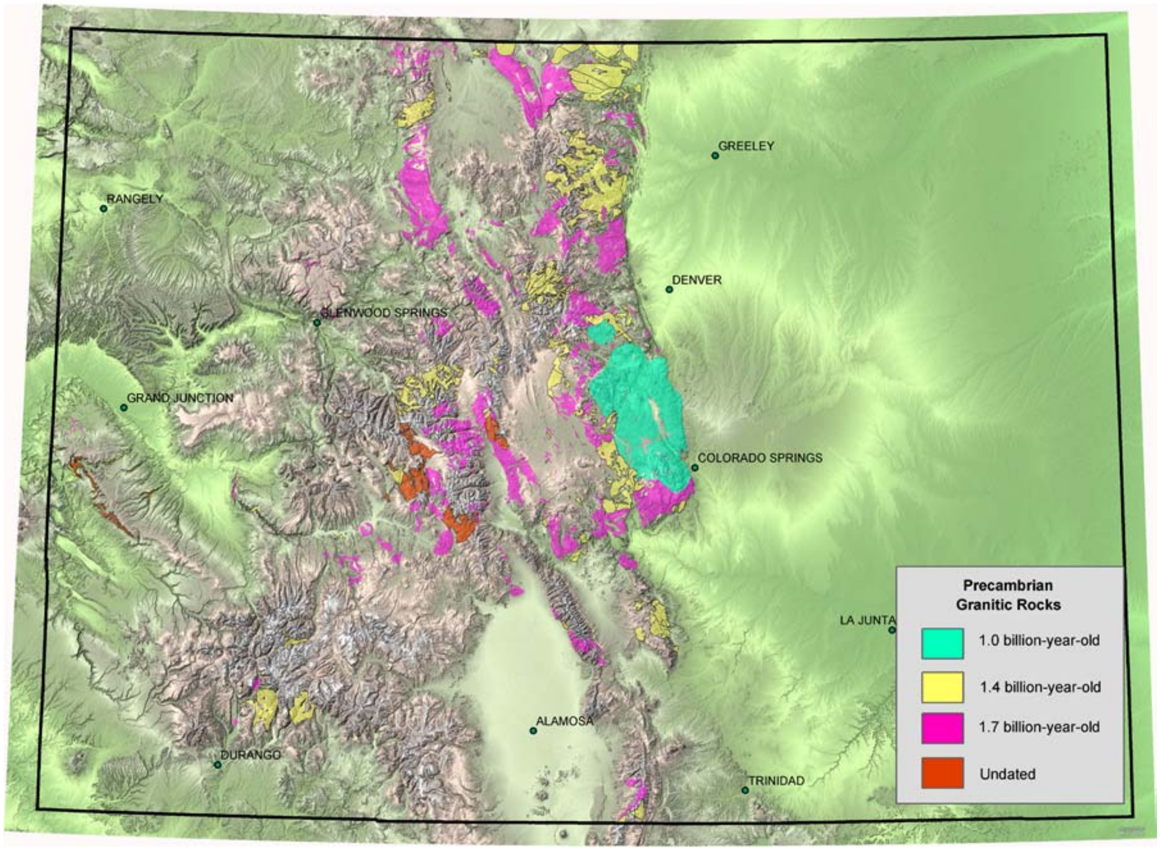

Figure 10. Distribution of Precambrian granitic batholith outcrops in Colorado. From Matthews and others (in prep.); modified from Tweto (1979).

it was at the equator, and by mid Permian time it was in the northern Horse Latitudes (i.e., 30 to 35° North Latitude). At the end of this era, Colorado was a relatively flat, low-lying region with an arid or semi-arid climate.

The surficial distribution of lower and middle Paleozoic strata in Colorado is shown in Figure 11. Descriptions of the five geologic periods contained within this time interval – the Cambrian, Ordovician, Silurian, Devonian, and Mississippian – are discussed in the following paragraphs.

Cambrian Period (543 to 490 million years ago) – Sea level rose about 500 million years ago as marine waters moved into Colorado from the west and south, depositing sands and pushing the shoreline ahead of the rising sea. As a result, Cambrian sandstones (Tintic and Ignatio Formations, Lodore Sandstone, Sawatch Sandstone) are generally younger as one travels east across the state (Myrow et al., 1999). These shoreline and near-shore sandstones are well exposed in the Sawatch Range, Glenwood Canyon, and near Red Cliff in central Colorado, at Baker's Bridge north of Durango, and along Highway 24 northwest of Manitou Springs. During the Cambrian Period, Colorado was close to the equator.

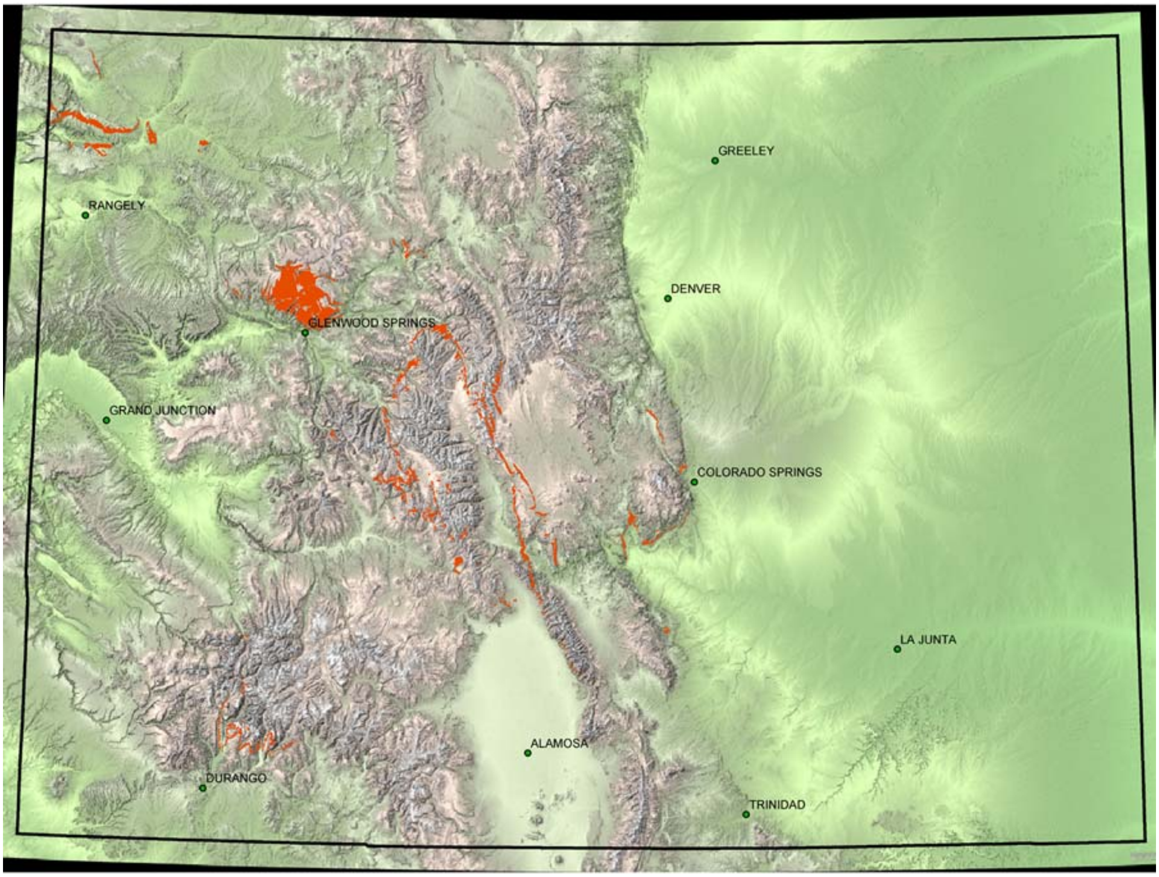

Figure 11. Distribution of lower and middle Paleozoic (Cambrian through Mississippian) rock outcrops in Colorado. From Matthews and others (in prep.); modified from Tweto (1979).

Ordovician Period (490 to 443 million years ago) – The Ordovician Period began with the shallow seas still covering much of Colorado and depositing carbonate rocks, predominantly dolomite with some limestone (Manitou Formation). A period of subaerial weathering and erosion preceded a sea level rise in late Ordovician time that deposited sands in central Colorado (Harding Sandstone). As sea level rose again just before the beginning of the Silurian Period, limestone (Fremont Limestone) was deposited in the warm tropical seas. During the Ordovician Period, North America shifted slowly southward, placing Colorado just south of the equator.

Silurian Period (443 to 417 million years ago) – Although sea level rose to one of its highest points during the Silurian Period, the extensive sediments that were deposited were soon eroded during a drastic drop in sea level that exposed the land. The only known exposure of Silurian rocks in Colorado are pieces of limestone from northern Colorado, from an area that was thought to have only Precambrian rocks. The rocks consist of blocks of fossiliferous limestone within diatremes, hosted in Precambrian rocks. Diatremes form by gaseous explosions in volcanic pipes. It is thought that these explosions broke off large blocks of overlying rock, which fell down into the pipe. Deep underground, these foundered blocks of Silurian rocks were protected from the erosion that removed the Silurian layers still exposed at the surface. Located only rarely along the Front Range near the Wyoming border, these are the only Silurian rocks found for 300 mi (482 m) in any direction.

Devonian Period (417 - 354 million years ago) – The withdrawal of the seas during the Silurian Period caused Colorado to be above sea level early in the Devonian Period, and many of the previously deposited rocks were subsequently eroded. Rocks deposited later in the Devonian Period reflect a relatively slow rise in sea level. During the late Devonian Period, a marine embayment covered most of the central and southwestern portions of Colorado, where first sandstone (Parting Sandstone) and then carbonate rocks (Dyer Formation) were deposited. The Parting Sandstone is generally well cemented with silica, and was named because it contains thin shale beds (i.e., partings) that made it easy to recognize.

Mississippian Period (354 - 323 million years ago) – Sea level peaked for the second time in the Paleozoic Era during early Mississippian time. Colorado was once again completely covered by shallow seas, which resulted in the deposition of a great thickness of limestone across the state (Leadville Limestone – equivalent to the Madison and Redwall Limestones elsewhere). About this time, approximately 340 million years ago, Colorado was part of the northern super-continent Laurasia and still lay slightly south of the equator. The Leadville Limestone is a well-known host rock for many ores that were emplaced much later during the Cenozoic Era, notably those around the town of Leadville.

Sea level dropped at the close of the Mississippian Period and Colorado once again became relatively dry land. During this time, the limestone dissolved and caves formed, creating an irregular land surface known as karst. Weathering of Leadville Limestone formed an uneven, karsted terrain (Figure 12). Often, this karst is overlain by a soil with a characteristic red color, known as terra rosa ("red land"). This weathered unit lies at the boundary between the strata of the Mississippian and Pennsylvanian periods. Remnants of this paleosol are best preserved in southwestern Colorado.

Pennsylvanian Period (323 - 290 million years ago) – The Pennsylvanian Period in Colorado began with the return of shallow seas, depositing fine-grained sands and black, marine shales (Molas Formation, Belden Shale). Conditions changed dramatically after these units were deposited due to the uplift of two north-south mountain ranges, the Ancestral Front Range and the Ancestral Uncompahgre Range. These mountains shed sediment into basins between the ranges and formed aprons and wedges of coarse sediment on both flanks of the two ranges. Because it is rare to find faults and folds formed during this time, the distribution of these coarse sediments is the primary evidence for this particular mountain-building event in Colorado.

Examples of the coarse, reddish sediment shed off the east side of the Ancestral Front Range are well displayed in the Flatirons near Boulder, Red Rocks and Roxborough state parks near Denver, and the Garden of the Gods near Colorado Springs (Fountain Formation). Similar sediments that were shed off the west side of the Ancestral Front Range are prominent between Vail and Avon along I-70 (Minturn Formation). In south-central Colorado, these sediments form the Sangre de Cristo Formation. Exposures of sediment shed from both sides of the Ancestral Uncompahgre Mountains appear in the striking red cliffs of Animas Canyon north of Durango (Hermosa Group) and the scenic Maroon Bells near Aspen (Maroon Formation) (Figure 13).

Figure 12. Partially filled cave and sinkhole in the Mississippian Leadville Limestone, exposed near Aspen. Photo from Maslyn, 1976.

The rising mountains blocked the surrounding seas and isolated a part of the sea between the two ranges, forming a restricted evaporite basin known as the Colorado Trough. Evaporation exceeded water inflow and caused the basin to be saturated by salts, which precipitated as solid minerals. The stark, desolate appearance of the hills along Interstate 70 between Eagle and Gypsum is caused by the presence of gypsum and other salts (Eagle Valley Evaporite). These evaporites are highly contorted as a result of extensive, solid-state flowage and deformation that occurred much later, during Cenozoic time. There were also algal mounds growing along the flanks of this evaporite basin, remnants of which can be seen near Minturn and Meeker.

The Paradox Basin in southwestern Colorado contains Pennsylvanian evaporite strata formed through a similar process. In this basin, thousands of feet of gypsum, salt, and potash were deposited in another restricted arm of the sea. These evaporites are interbedded with limestones and black shales documenting dozens of depositional cycles (Pinkerton Trail, Paradox, and Honaker Trail Formations). Algal mounds are reservoirs for some of the basin's oil fields.

Although the Ancestral Rockies uplifted much of the area, shallow seas still repeatedly invaded the lowland areas. Marine fossils document as many as 20 marine cycles in the strata that crop out west of Vail and in the subsurface in the Denver Basin of eastern Colorado. These cyclic deposits are the result of continental glaciers in the Southern Hemisphere that repeatedly stored and released large volumes of water, causing the relatively rapid fluctuation of sea level during the Pennsylvanian Period.

The surficial distribution of Pennsylvanian and Permian rocks in Colorado is shown in Figure 14.

Figure 13. The Maroon Bells near Aspen, composed of gently dipping beds of Pennsylvanian Maroon Formation.

Permian Period (290 – 251 million years ago) – As the Permian Period began, erosion continued to wear away the Ancestral Rocky Mountains and fill the intermontane basins with coarse sediment. Several of these formations (Fountain, Maroon, Sangre de Cristo, Cutler) are Pennsylvanian to Permian in age. The humid conditions of Pennsylvanian time were replaced by a more arid climate in the Permian. The sea level dropped, exposing more dry land in Colorado. On the land, large dune fields dominated the newly exposed landscape (Weber Sandstone, Lyons Sandstone). In the western part of the state, limestone, sandstone, and shale were still being deposited in a shallow sea (Phosphoria, Park City, and State Bridge Formations).

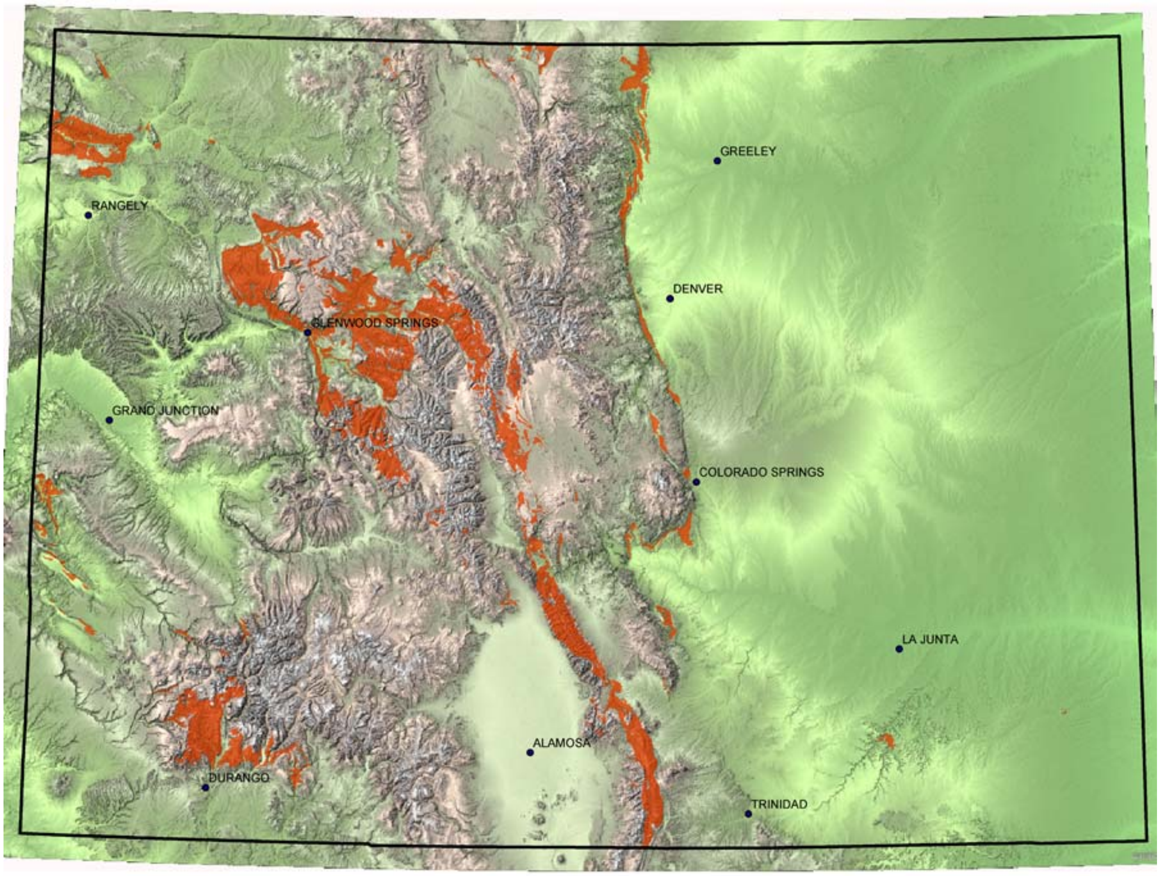

Figure 14. Distribution of Pennsylvanian and Permian rock outcrops in Colorado. From Matthews and others (in prep.); geology from Tweto (1979).

As the Permian Period ended, Colorado was a relatively flat, low-lying region with an arid or semi-arid climate. All of the continents had coalesced to form the supercontinent of Pangaea, and Colorado was on the edge of the shallow sea with sea level continuing to drop. The Permian Period ended with a profound mass extinction. In stark contrast to its vibrant beginning in the Cambrian Period, at least half of the known families of both marine and terrestrial organisms died out at the close of the Paleozoic Era. Scientists estimate that in this period of mass extinction, 75 percent of the amphibian families and more than 80 percent of the reptile families disappeared.

The Mesozoic Era

The Mesozoic era, encompassing three distinctly different geologic periods – Triassic, Jurassic, and Cretaceous – spans the time period from 248 to 65 million years ago. Dinosaurs appeared and were the dominant land life forms at this time. During the first two periods, Colorado was a land area of low relief with a persistent warm, dry climate. During much of the third and final period, a vast seaway covered the western interior of the North American continent, including the entire state at its fullest extent. The end of the Mesozoic Era was marked by the initial rise of the modern-day Rocky Mountains. The seas of the western interior seaway retreated for the last time, coincident with the emerging mountain ranges, and terrestrial basins dominated the state's landscape through the ensuing Cenozoic era. As with the Paleozoic, the Mesozoic Era ended with a mass extinction, this time of the dinosaurs.

Mesozoic rocks lie at, or very near, the surface of about 35 percent of the state and under an even greater area in the subsurface. A typical exposure from west-central Colorado is shown in Figure 15. Overall, these rocks contain important energy resources including oil, natural gas, and coal. Several of the formations also contain important industrial minerals and ground water, as well as uranium, radium, and vanadium in western Colorado.

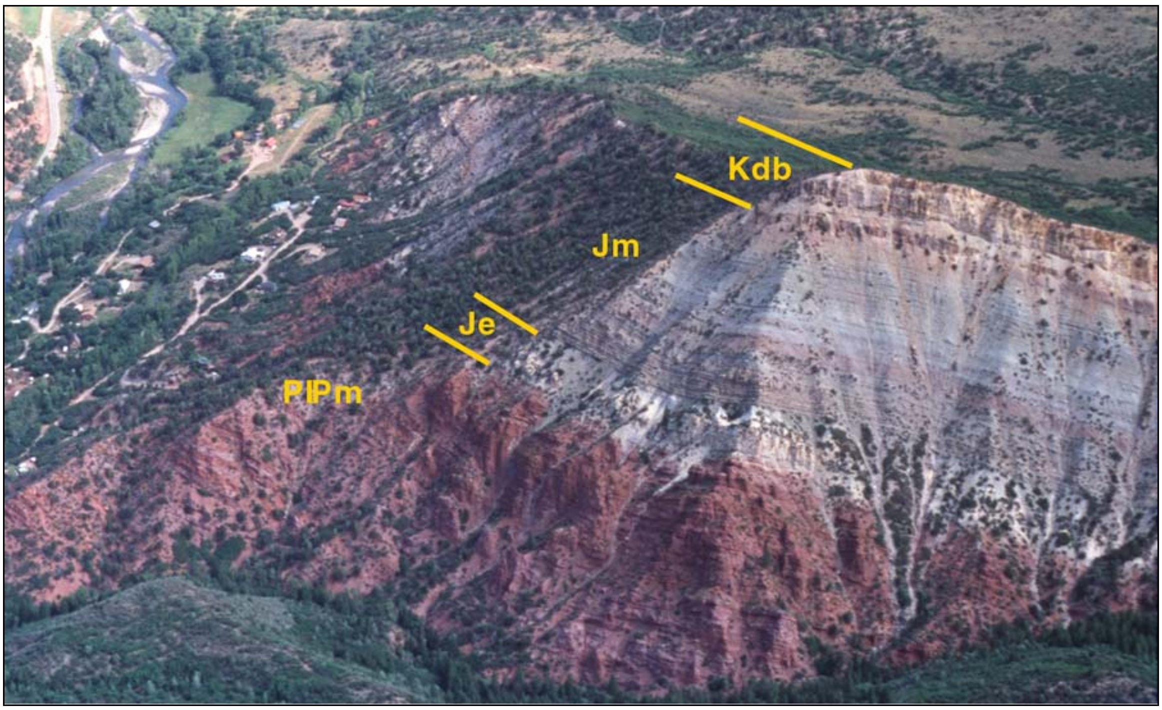

Figure 15. Outcrop of Mesozoic rocks overlying upper Paleozoic rocks in west-central Colorado, near Carbondale. (IPPm = Maroon Formation; Je = Entrada Sandstone; Jm = Morrison Formation; Kdb = Burro Canyon Sandstone and Dakota Group). Photo by David Noe and Jonathan White, Colorado Geological Survey.

Triassic Period (251-206 million years ago) – The sediments deposited during much of the Triassic Period are remarkably similar in composition and general character throughout the United States. Triassic red beds impart their hues to many western Colorado landscapes. They are comprised of fine-textured sandstones, siltstones, and shales that were deposited in mudflats, on alluvial plains, and in dune fields adjacent to eroding highlands. The red beds sometimes interfinger with limestone, halite, and gypsum – probably the result of brief incursions of restricted marine waters from the north onto flat-lying coastal and alluvial plains.

Triassic red beds are exposed along much of the eastern edge of the Front Range Uplift (Lykins Formation) and form part of a strike valley between two steeply dipping, hogback ridges of Permian and Cretaceous age. These valleys extend intermittently from Douglas County north to the Wyoming state line. In western Colorado, the near-horizontal Triassic red beds (Moenkopi Formation, Chinle Formation) are easily eroded, and generally appear as debris-covered slopes between resistant ledges.

The surficial distribution of Triassic and Jurassic rocks in Colorado is shown in Figure 16.

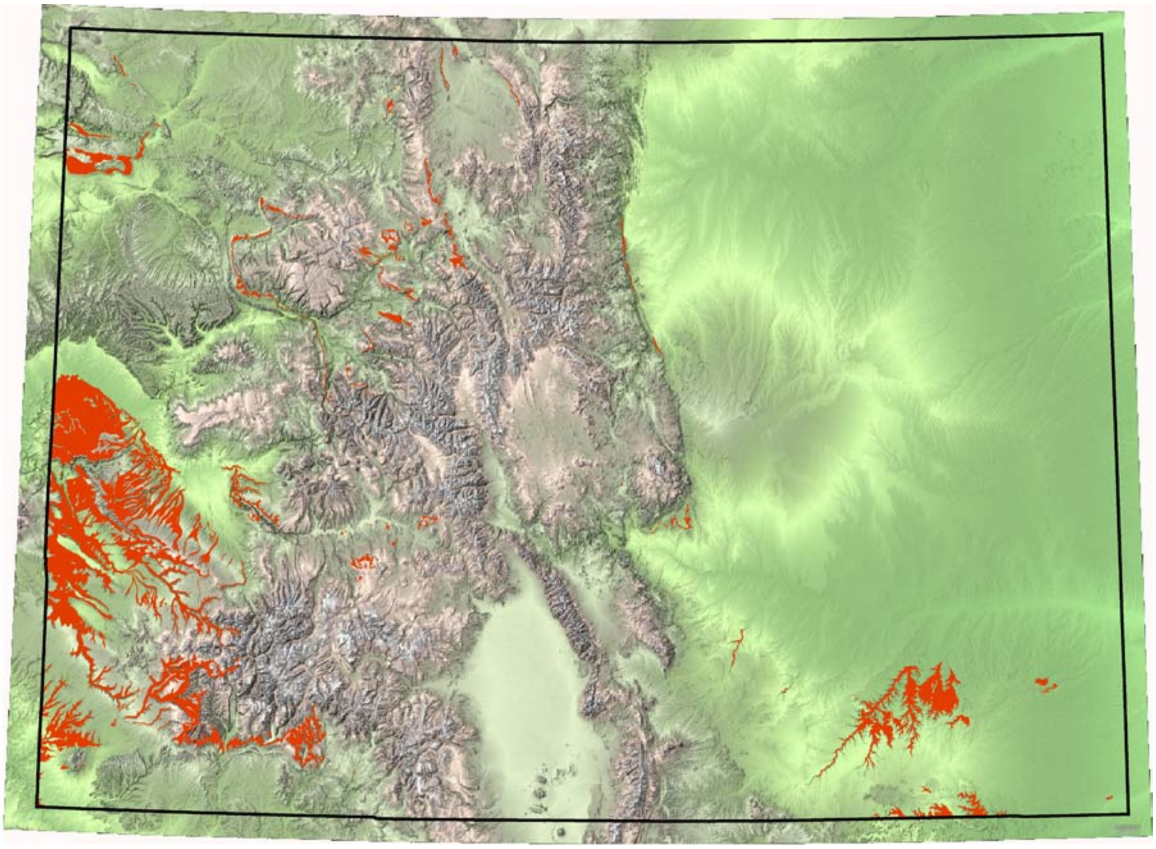

Figure 16. Distribution of Triassic and Jurassic rock outcrops in Colorado. From Matthews and others (in prep.); modified from Tweto (1979).

Jurassic Period (206-144 million years ago) – The deposition of red beds in Colorado was followed in late Triassic and early Jurassic time by deposition of widespread dune sands in a warm, desert climate. This climate change resulted from the separation of the North American plate from Africa. Colorado and North America drifted north out of tropical latitudes into the Horse Latitudes. The Jurassic dune sandstones (Entrada Sandstone) feature large-scale cross bedding, and form spectacular, light colored cliffs throughout much of western Colorado.

Deposition of sand dunes was interrupted four times during the first 45 million years of the Jurassic Period, when marine waters advanced from the northwest. This seaway and its deposits (Curtis, Summerville, and Ralston Creek Formations) covered parts of northwestern and north-central Colorado. Remnants of the Ancestral Rocky Mountains remained just above sea level in the southern and central areas of the state.

After final retreat of the Jurassic seaway, the entire Rocky Mountain region became a vast, continental lowland of lakes, swamps, dunes, and braided and meandering streams. The resulting deposits (Morrison Formation) are similar in appearance and thickness throughout this region. This great lowland area and its deposits covered a ten-state area in the western interior of the United States and southern Canada.

The Morrison Formation yields one of the richest fossil assemblages of dinosaurs on the North American continent including *Stegosaurus* (the state fossil of Colorado) (Jenkins and Jenkins, 1993). In Colorado, this formation also provides the continent's largest assemblage of dinosaur trackways at a site located south of La Junta along the Purgatoire River. The site has more than 1,300 tracks contained in roughly individual 100 trackways, which are exposed in outcropping limestone layers near the river (Lockley et al., 1999).

Cretaceous Period (144 to 65 million years ago) – The Cretaceous Period began in Colorado with the lowland topography inherited from the preceding Jurassic. Sea level began rising, with the Gulf of Mexico approaching from the south and the Arctic Ocean migrating from the north. Eventually, these water bodies joined, forming a vast, relatively shallow seaway across the western interior of North America (Figure 17).

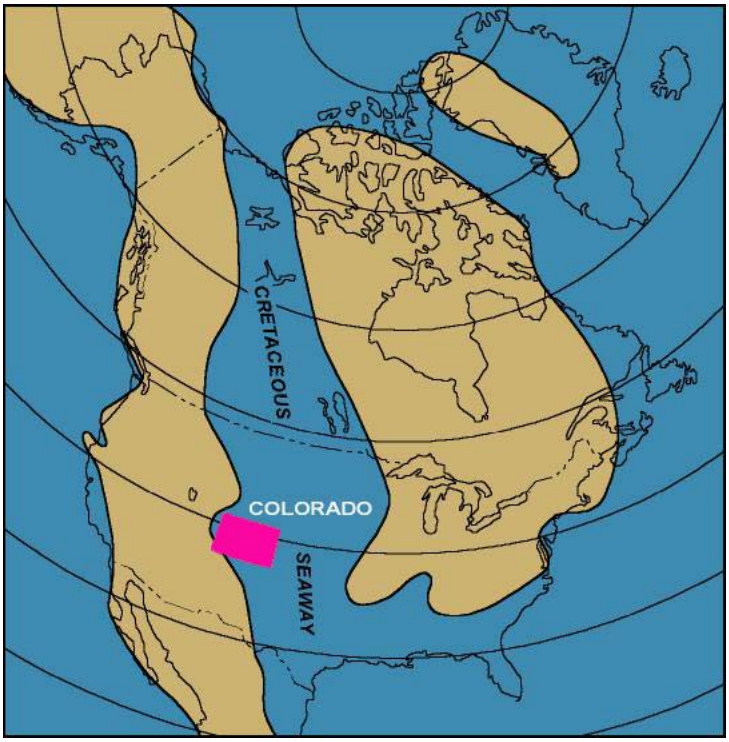

Figure 17. The Cretaceous Western Interior Seaway in one of its many shoreline configurations, showing Colorado's location. From Matthews and others (in prep.); modified from Gill and Cobban (1973).

Widely fluctuating geologic conditions resulted in a complex intertonguing of marine and terrestrial deposits during this period. Uplift of the Sevier thrust belt to the west in Utah contributed sediment pulses, and volcanic activity in the Elkhorn Mountains of southwestern Montana contributed ash fall deposits (bentonites). The entire area from central Utah through western Colorado was a slowly subsiding foreland basin, allowing thick accumulation of sediments. A complicated interplay between basin subsidence, sediment inflow, and sea-level fluctuations affected the shifting of the shorelines.

The beginning of the Laramide mountain-building event ushered in the final retreat of western interior seaway, and a return to terrestrial conditions that began during the last few million years of the Cretaceous Period and continue today (Weimer, 1996; Raynolds, 2002). The surficial distribution of Cretaceous rocks in Colorado is shown in Figure 18.

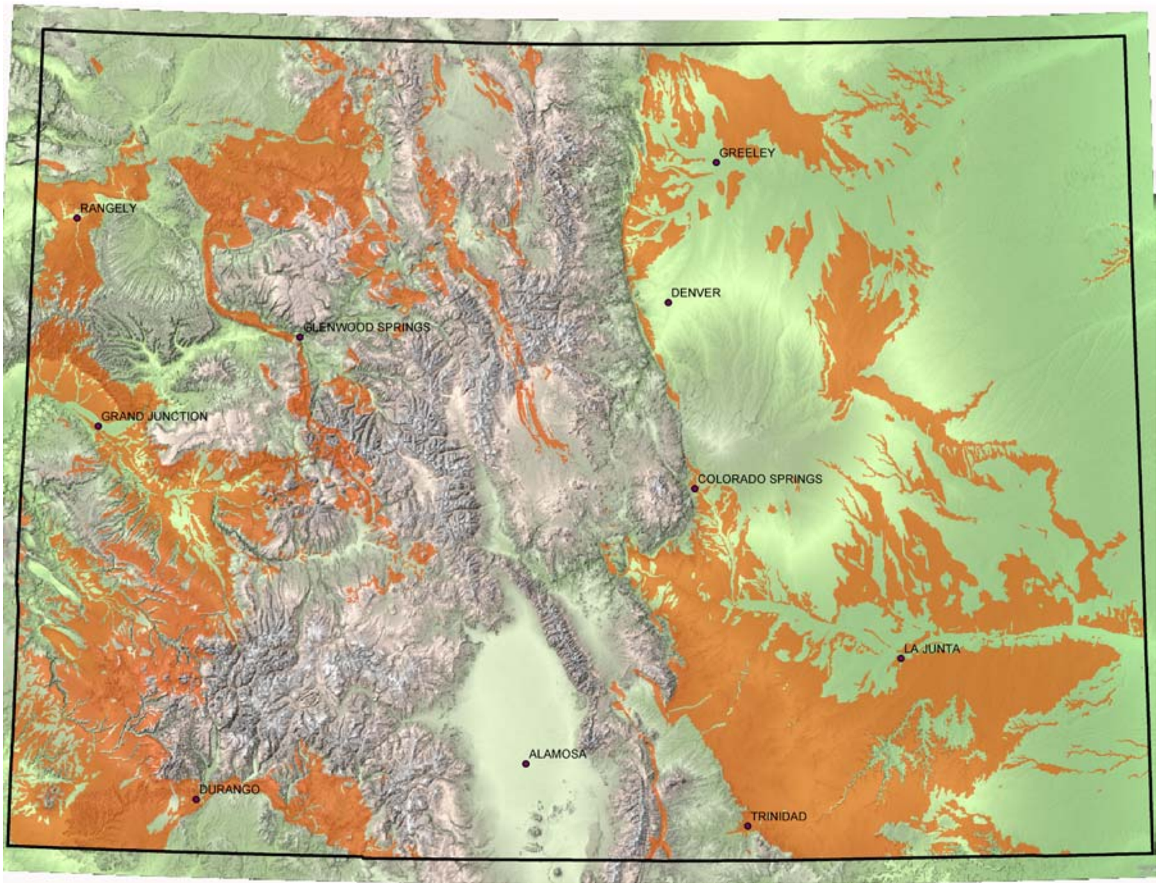

Figure 18. Distribution of Cretaceous rock outcrops in Colorado. From Matthews and others (in prep.); modified from Tweto (1979).

The oldest Cretaceous rocks in Colorado are represented by the Dakota Group, which was widely deposited as a complex of beach, deltaic, estuarine, and nearshore marine sediments. Rocks from this group are found throughout Colorado. Sandstones from the Dakota Group are important oil, gas, and ground water reservoirs. These sandstones are resistant to erosion and form the prominent Dakota Hogback ridge along the eastern foothills of the Colorado Front Range. Elsewhere, where Dakota sandstones are relatively horizontal, they typically form a rimrock or a distinctive mid-slope ledge.

The Dakota Group is overlain by thousands of feet of marine shale and limestone, which were deposited as the seaway deepened and marine waters advanced across the state. In eastern Colorado, the widespread Graneros Shale, Greenhorn Limestone, Niobrara Formation, and Pierre Shale represent this episode. The Pierre Shale is the thickest of these formations, being over 8,000 ft (2,438 m) thick near Boulder (Scott and Cobban, 1965). It underlies most of eastern Colorado and has an extensive outcrop area. These rocks are easily weathered and eroded, and they most often form grass-covered, low-relief topography. They contain discrete and laterally extensive bentonite beds that are typically less than one foot thick.

At this same time, the intertonguing marine and terrestrial strata of the Mancos Shale and the Mesaverde Group were deposited in the western part of the state. The Mancos Shale is essentially equivalent to the Graneros Shale, Greenhorn Limestone, Niobrara Formation, and Pierre Shale, although it is siltier and less calcareous as a consequence of being deposited nearer to the ancient shoreline the was located to the west. The Mesaverde Group records the repeated incursion of terrestrial sediments from the highlands in Utah, which prograded the shoreline eastward across Colorado in pulse-like steps. Extensive deposits of interbedded sandstone, shale, and coal, resulting from accumulation of organic matter in marshes and lagoons, are widespread in the Mesaverde Group.

Beginning about 70 million years ago, the long history of deposition and subsidence of the Mesozoic Era came to an end with the Laramide mountain-building episode. At this time, the entire western continental interior rose regionally. The seas of the late Cretaceous Seaway retreated for the last time and were replaced by the emerging mountain ranges and terrestrial basins that would dominate the ensuing Cenozoic Era. Along the margin of the retreating sea, shoreline deposits in central and eastern Colorado formed the Fox Hills Sandstone. A return to continental deposition is recorded in the overlying, interbedded sandstone, shale, and coal of the Laramie and Lance Formations. Finally, regional uplift and volcanism, coupled with erosion of those developing highland areas, resulted in the deposition of coarse- to fine-grained terrestrial sediments of the Arapahoe, Denver, and Dawson Formations in the age-equivalent Denver Basin (Raynolds, 2002).

Much like the Paleozoic, the Mesozoic Era ended with a massive, worldwide extinction of many species, including all of the larger dinosaurs. In the 1940s, South Table Mountain west of Denver was the first place in the world where the boundary between the Cretaceous and Tertiary Periods was described in terrestrial rocks, with only dinosaur bones below and only mammals above (Brown, 1943). In southern Colorado, the K/T boundary was preserved in a clay layer in swamps that later became the rich coal deposits of the Raton Basin. The layer contains anomalously high amounts of iridium (in fact, the highest iridium content ever measured in continental rocks), a rare element in terrestrial rocks but common in meteorites. It contains soot and fragments of minerals that show evidence of being strongly shocked by impact. These rocks yielded physical and chemical data that played a key role in proving that the Chicxculub crater in Mexico's Yucatan Peninsula was the source of meteor impact debris that affected the planet (Izett, 1990; Bohor et al., 1993; Krogh et al., 1993; Powell, 1998).

The Cenozoic Era

In comparison to those eras that preceded it, the Cenozoic Era is relatively short and is divided into two very unequal periods of time – the Tertiary and Quaternary. Over 63 million years are classified as the Tertiary Period, while only the last 1.8 million years belong to the Quaternary Period. Because the Cenozoic Era is the most recent span of geologic time, much is known about it. The animals and plants that evolved during this Era are ancestors of the fauna and flora of our present environment. In Colorado, mountain building, rifting, volcanic eruptions, glaciers, widespread erosion, and basins filling with sediment characterized the era. The Rockies are not Precambrian mountains, but rather Cenozoic mountains made of Precambrian rocks.

Tertiary Period (65 to 1.8 million years ago) – The Tertiary Period in Colorado includes two mountain-building events, three igneous events, and the deposition of thick, terrestrial sediments. The Laramide mountain-building event, which began in the late Cretaceous, continued for the first 25 million years of the Tertiary. Thousands of feet of uplift and downwarp in many areas across the state formed ranges and basins, some of which have been exhumed to form elements of Colorado's spectacular, present-day scenery. Dramatic products of this period include the fold and fault belt from Lyons to Fort Collins, Boulder's Flatirons, the hogbacks along the Front Range, the Garden of the Gods in Colorado Springs, the Hogback Monocline near Durango, and the Grand Hogback near Glenwood Springs (Figure 19). The monoclines of the Colorado National Monument in Grand Junction and Dinosaur National Monument of northwestern Colorado also emerged at this time.

Igneous activity accompanied the Laramide mountain-building event. Intrusive rocks of Laramide age (72-50 million years old) are found throughout the northeast-trending Colorado Mineral Belt. Although volcanoes are not preserved, we know that magma reached the surface and built volcanoes because volcanic fragments are found in age-equivalent sedimentary rocks in the Denver Basin (Denver Formation) and in Middle Park (Middle Park Formation). Near Golden, andesitic flows that once flowed down a paleovalley are preserved as rimrock on North and South Table Mountains, and are a spectacular example of reverse topography.

Laramide mountain building ended much as it began – with activity tapering off at different times in different places. About 38 million years ago, a combination of beveling by erosion and burying by deposition reduced the entire area to a broad undulating surface of low relief. Former mountains were buried in their own debris. Intensive erosion during and following the Laramide is recorded in thick sequences of Tertiary sedimentary rocks in the basins between mountain ranges and on the plains (Wasatch, Coalmont, Denver, Dawson, Browns Park, Troublesome, White River, Arikaree, and Ogallala Formations) (Figure 20). Although the post-Laramide erosion surface has been uplifted, faulted, and dissected by stream and glacial erosion in ensuing time, remnants of the eroded surface are preserved as pediments in many areas of central Colorado.

Widespread volcanic activity occurred about 36 million years ago and continued for 10 million years, depositing volcanic ashes, lava flows, and lahars (volcanic mudflows) (Figure 20). The San Juan Mountains represent one of the larger volcanic regions, containing several large calderas or basin-shaped volcanic depressions. Volcanic rocks were deposited in many areas of

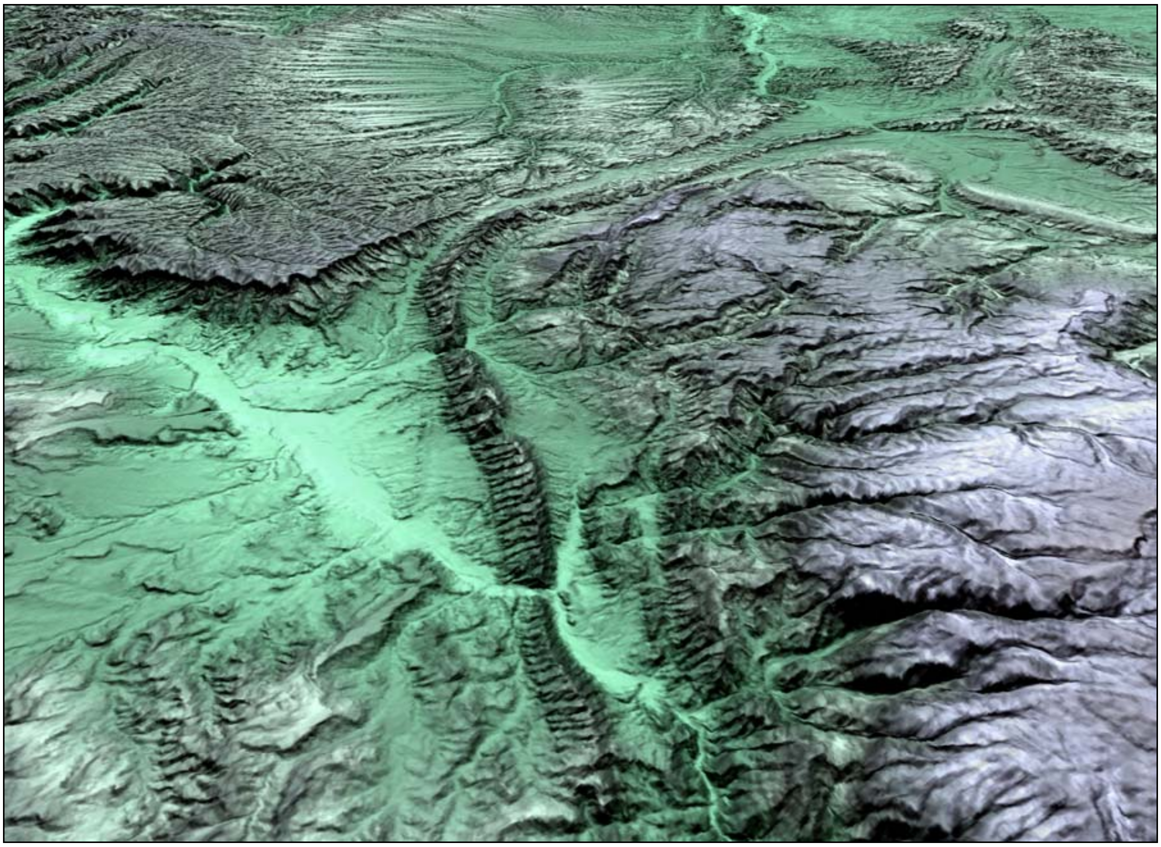

Figure 19. Digital elevation model of the Laramide-age Grand Hogback near Glenwood Springs, located between the Piceance Basin (to the left) and White River Plateau (to the right). From Matthews and others (in prep.); 100-meter DEM data from David Catts, U.S. Geological Survey.

the state on the low-relief, post-Laramide erosion surface. At its maximum extent, this great coalescing sheet of volcanic outpourings covered half of the state (Figure 21).

During late Cenozoic time, the Rocky Mountains were again uplifted and deformed, producing Colorado's present topography of block-faulted mountains and basins, plateaus, and high plains. The previous andesitic igneous activity was replaced by basaltic volcanism as Colorado's crust pulled apart and extended to the east and west. As extension progressed, the crust was broken into many blocks that rose and fell, creating mountains and basins. Many of these fault blocks were reactivated older Precambrian, late Paleozoic, or Laramide structures. Individual fault displacements of as much as 20,000 ft (6,096 m) occurred between the San Luis Valley and the Sangre de Cristo Range during late Cenozoic time. This activity continues today, as evidenced by recent, active faulting and accompanying earthquakes and fairly recent, basaltic eruptions (Giegengack, 1962; Kirkham and Rogers, 2000; Naeser et al., 2002; Steven, 2002; Widmann et al., 2000; 2002).

The most dominant, late-Tertiary structural feature in Colorado is the Rio Grande Rift, which can be traced nearly 500 mi (805 km) from Mexico to Colorado, entering the state through the San Luis Valley and continuing to the north in a series of segments to the vicinity of Steamboat

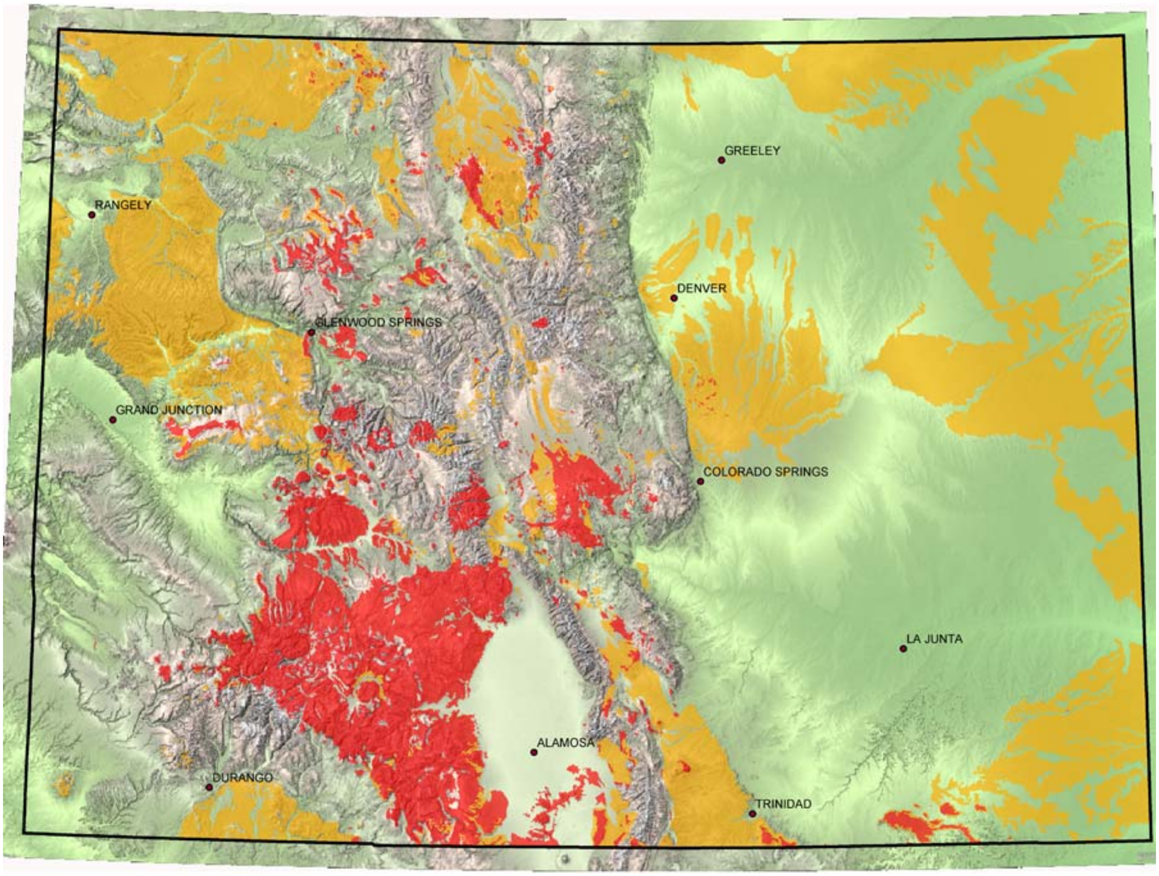

Figure 20. Distribution of Tertiary sedimentary (gold) and volcanic (red) rock outcrops in Colorado. From Matthews and others (in prep.); modified from Tweto (1979).

Springs. The rift is a series of north-trending, down-faulted basins or grabens, flanked on one or both sides by up-faulted mountain ranges. The most outstanding grabens or structural valleys of the rift are the San Luis Valley, the Upper Arkansas Valley from Salida to Leadville, and the Blue River Valley from Silverthorne to Kremmling. Examples of adjacent faulted mountain blocks include the Sangre de Cristo, Mosquito, and Williams Fork ranges on the east flank of the rift and the Sawatch, Tenmile, and Gore ranges on the west. The subsiding grabens acted as sedimentary basins and collected thousands of feet of debris that were shed as the rising mountains were subjected to the forces of erosion (Naeser et al., 2002; Steven, 2002).

Continued uplift and accelerated canyon cutting and erosion characterized the end of the Tertiary Period. The Royal Gorge, Clear Creek, Big Thompson, Cache la Poudre, Glenwood, and the Black Canyon of the Gunnison are a few of the spectacular canyons cut during the last five million years. The Colorado River formed Glenwood Canyon as it eroded into the southern flank of the rising White River Uplift. Most of the steep inner walls of the canyon were carved during the past three million years (Kirkham et al., 2001). At the same time, the Colorado River was carving the Grand Canyon downstream in Arizona.

In certain areas of Colorado, particularly near Eagle, Glenwood Springs, Carbondale, Buford, and Paradox, canyon incision by the major rivers created an imbalance of overburden pressure

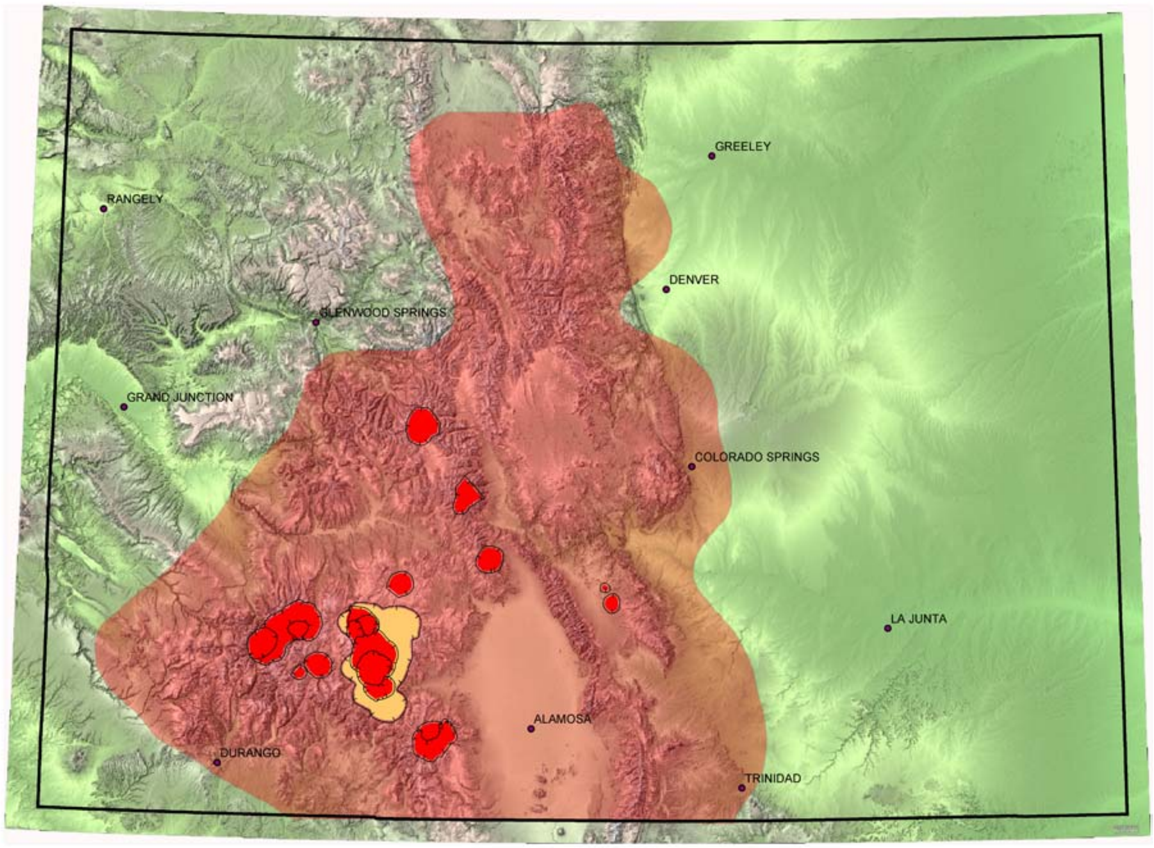

Figure 21. Distribution of calderas (red and gold ovals with hachures) and interpreted original extent of Tertiary volcanic rocks (transparent orange shading) in Colorado. From Matthews and others (in prep.); geology from Steven (1975) and Lipman (2000).

on the underlying formations. Buried evaporite deposits of Pennsylvanian age flowed laterally and upward and reached the ground surface along the canyons, resulting in of evaporite diapirs and river-centered anticlines along those valleys. In the surrounding areas, nearly 4,000 ft (1,219 m) of regional collapse occurred due to the evacuation, outflow and migration, and near-surface dissolution of halite and gypsum. There is evidence that this process is ongoing today (see Kirkham et al., 2002). For information about the impacts to humans from evaporite dissolution and evaporite tectonism in these areas, see the "Geology and Water Quality" and "Earthquakes and Seismicity" sections in Part II of this paper (Noe et al., 2003). In addition, there are related technical papers in the following chapters of this volume: "Faulting and Earthquake Hazards" and "Expansive and Collapsible Soil and Bedrock."

Quaternary Period (1.8 million years ago to present) – The Quaternary period, the most recent period of geologic time, has been dubbed the "Ice Age," as great continental glaciers covered most of Canada and much of the northern United States at certain times. Although the continental ice sheets did not extend into Colorado, several episodes of cooler climate produced alpine glaciers in many of the state's mountain ranges. During this time period, glaciers, water, wind, gravity, volcanoes, and tectonics modified the Tertiary landscape to produce the recent sediment deposits and awe-inspiring landforms we see in Colorado today.

These deposits and landforms are significant in that many engineering-geologic projects involve the characterization and evaluation of Quaternary deposits. Although they are widespread across the state, Colorado's Quaternary deposits are rarely grouped and described in a time-stratigraphic context. Notable exceptions include the glacial deposits in the mountains (e.g., Bull Lake and Pinedale) and the alluvial-terrace deposits along the Front Range Piedmont (e.g., Nussbaum, Verdos, Slocum, Louviers, Broadway, Piney Creek, and Post-Piney Creek). More commonly, these recent deposits are grouped and described in terms of their geomorphic landforms and the physical processes that created them. The following paragraphs describe these landforms and processes, many of which are active today, and their associated deposits.

Glaciers – A glacier is a moving body of ice formed by the accumulation, compaction, and recrystallization of snow. Before their disappearance 12,000 years ago, large glaciers thousands of feet thick filled valleys in the mountainous regions of Colorado (Figure 22).

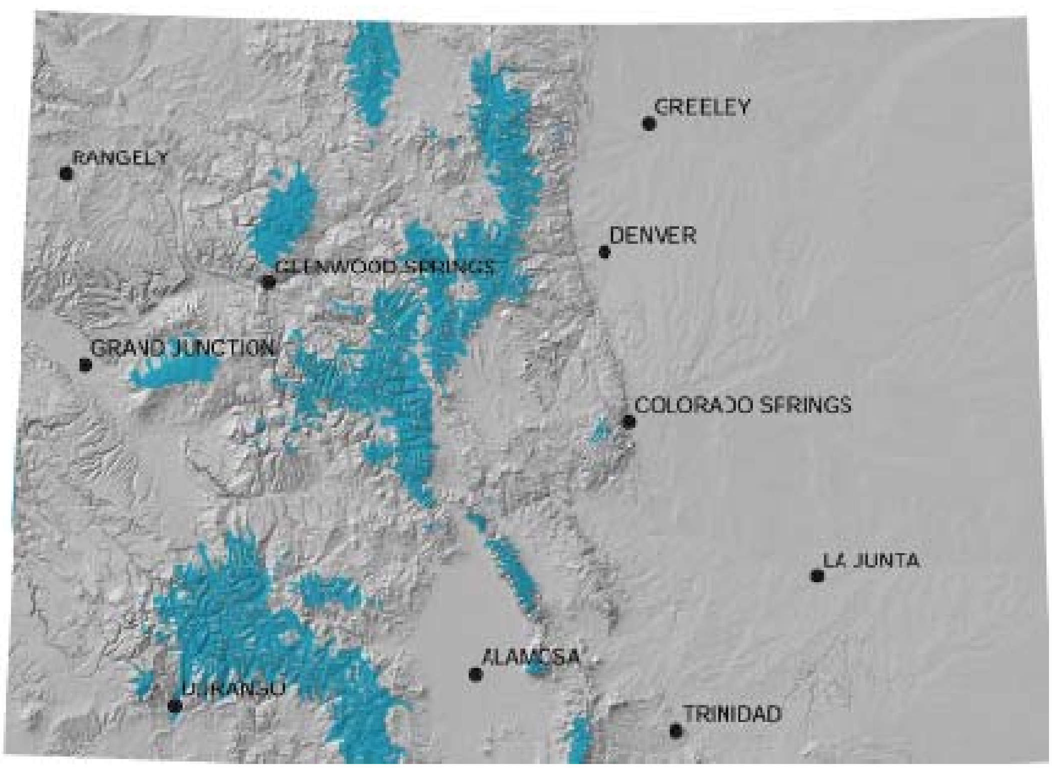

Figure 22. Maximum extent of alpine glaciers in Colorado. From Matthews and others (in prep.); geology from Meierding and Birkeland (1980).

Although these massive glaciers are long gone, they left behind thousands of cirques, arêtes and horns, tarns, oversteepended u-shaped valleys, polished and grooved rock surfaces, and moraines. Geologists are able to distinguish deposits in Colorado of at least three Quaternary ice ages. The two most recent glacial events peaked between 130,000 and 150,000 years ago and about 20,000 years ago. The age of the older Quaternary glacial deposits is harder to decipher in the state. Only a dozen or so modern glaciers are indicated on topographic maps in Colorado. Today's small glaciers are not remnants of the former large glaciers, but formed in sheltered mountainous landscapes in Colorado during the Little Ice Age, between 1200 and 1880 A.D.

Rock Glaciers – Active and inactive rock glaciers are common in many areas of the high Rocky Mountains. Their overall shape resembles that of ice glaciers. Large boulders and smaller stones typically comprise the surface, which commonly has arcuate ridges and lobes (Figure 23).

Figure 23. A rock glacier on the side of Mt. Sopris, near Carbondale.

Rock glaciers may contain ice at their cores, or may simply have interstitial ice within the rock mass that deforms, allowing flowage. These features may be a mile or more long and have steep, unstable fronts, which may exceed 300 ft (91 m) in height and stand at angles approaching 45 degrees. Movement rates for active rock glaciers are variable. Site-specific measurements range from less than 8 in (20 cm) per year in the Front and Sawatch Ranges to as much as 24 in (61 cm) per year in the Elk Mountains (Benedict, 1970).

Periglacial Features – The term "periglacial" is applied to processes and landforms associated with very cold climates in areas not permanently covered by snow or ice, regardless of age. In Colorado, periglacial features are common at high elevations – typically above 11,500 ft (3,505 m) – where prevailing temperatures are so low that the ground remains frozen for much of the year. The effects of repeated freezing and thawing, and the growth of ice masses in the ground, produce many unique alpine features. These include patterned ground such as rock bands, nets, circles, and polygons, and accumulations of large, angular blocks of rock known as "felsenmeer" (German for "rock sea"). A popular stopping point along Trail Ridge Road in Rocky Mountain National Park is Rock Cut, where a trail leading from the road provides easy access through felsenmeer and patterned ground.

During the summer season in the high country of Colorado, water is unable to percolate into an impervious layer of frozen ground below the surface. As a result an "active layer" of soil becomes supersaturated and creeps downslope. This process, called solifluction, can occur on slopes as gentle as two or three degrees. Where there is a well-developed mat of vegetation, a solifluction sheet may move downward in a series of well-defined lobes and form terrace-like features. Rates of downslope movement may vary depending on local conditions, but rates of about 2 in (5 cm) per year are typical for Colorado's Front Range (Benedict, 1970).

Sackungen – A sackung is a fault-like feature that is associated with post-glaciation collapse of a mountain, and not tectonic movements. During glaciation, the mountain valleys were so excessively eroded and oversteepened that there was inadequate lateral support for the valley walls when the glaciers melted. Subsequently, the mountains slowly bulged out into the valleys under their own weight and the force of gravity, and slow, vertical collapse occurred at or near the mountaintops. Small fissures and faults formed to accommodate this movement. The resulting surficial scarp, which nearly always faces uphill, is called a sackung (Figure 24). Sackungen are found along ridge crests in many glaciated areas of Colorado. When sackungen occur on both sides of a ridge, they form a crestal graben. Trail Ridge Road in Rocky Mountain National Park follows a crestal graben at one location. It is possible that large earthquakes on nearby faults could trigger episodic movements on sackungen and lead to large rock-slope failures.

Alluvial Deposits – During Quaternary time, Colorado's climate changes and glacial activity resulted in periods of high stream flow. The glacial meltwater accelerated ongoing erosional downcutting in several mountain valleys, and resulted in the deposition of alluvium in the stream valleys. Along the Front Range, well away from glaciated terrain, the through-flowing mountain streams and other local streams eroded away much of the Tertiary rock cover, exposing older sedimentary rocks and forming the Front Range Piedmont physiographic sub-province. The Quaternary alluvial deposits in Colorado (Figure 25) consist of numerous outwash plains and alluvial terraces, which record episodes of deposition and downcutting that date back to the ice ages, as well as modern floodplain deposits.

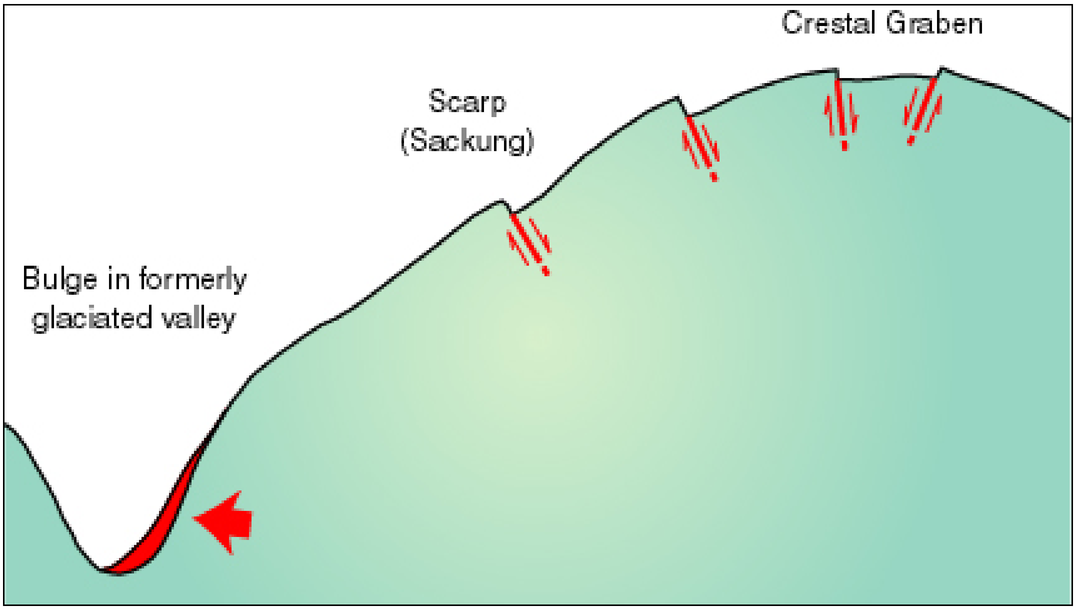

Figure 24. Schematic cross-section through a mountain, showing sackungen that develop as a result of spreading of the mountain mass towards its oversteepened, previously glaciated flanks. Modified from Varnes and others (1989).

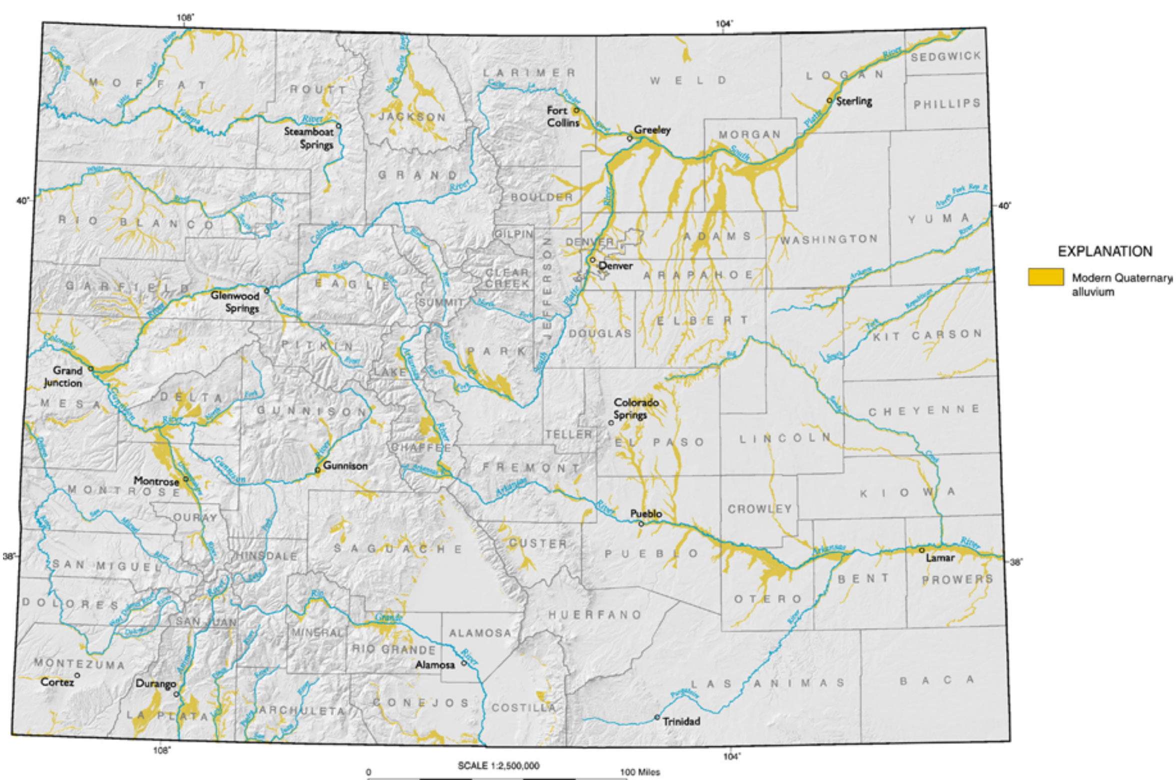

Figure 25. Distribution of Quaternary alluvial deposits in Colorado. From Topper and others (2003); modified from Tweto (1979).

Alluvial deposits in Colorado are quite varied in their thickness and composition as a function of local and upstream geologic and climatic conditions and the stream gradient. These sediments range from boulder and cobble deposits in the mountains and grade downstream into sandy and silty deposits on the plains and in the western valleys.

Eolian Deposits – Colorado's dry and windy climate during the Quaternary Period created widespread deposits of wind-blown material. These eolian deposits cover huge areas of the state's eastern plains. Other eolian deposits are located in southwestern Colorado in the Paradox Valley, in northwestern Colorado near the Little Snake River, in north-central Colorado along the eastern edge of North Park, and in the Great Sand Dunes National Park in the San Luis Valley of south-central Colorado (Figure 26).

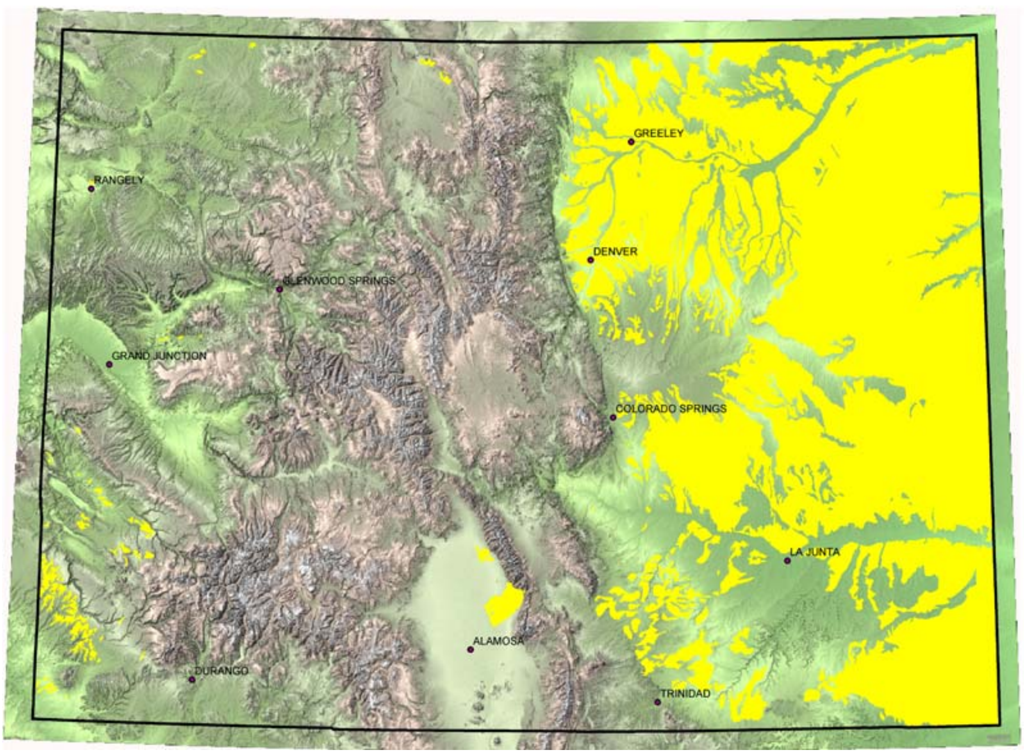

Figure 26. Distribution of Quaternary eolian deposits in Colorado. From Matthews and others (in prep.); modified from Tweto (1979) and Madole (1995).

The most extensive eolian deposits are found east of the Rocky Mountains on Colorado's high plains, where wind-laid deposits of loess and sand cover more than 30,000 mi^2 (77,700 km^2). The loess (silt) and sand make up 70 and 30 percent of the composition, respectively, of these deposits. The South Platte and Wray dune fields of northeastern Colorado are two of the state's largest dune fields, covering nearly 5,000 mi^2 (12,950 km^2). Here, during the last glacial period, winds blew predominantly from the northwest and caused the dunes to migrate in a southeasterly direction. Parabolic dunes predominate in this part of Colorado. Loess deposits reach a thickness of more than 150 ft (46 m) near Beecher Island, to the south of Wray, although a thickness of 6 to 15 ft (1.8 to 4.6 m) is more common elsewhere (Madole, 1995).

Caves – Colorado has more than 265 catalogued caves (Figure 27). These caves form from a variety of processes, including dissolution and erosion. Some contain evidence of early human habitation, while others have yielded significant and unusual animal remains. The largest caves are dissolved from limestone, usually Mississippian Leadville Limestone, which is thick, pure, and areally extensive. The largest of these, Groaning Cave in Eagle County, is over 10 mi (16 km) long. Two commercial cave operations, at Glenwood Caverns near Glenwood Springs and Cave of the Winds near Manitou Springs, have been hosting tours since the 1880s. Fairy Cave, a part of Glenwood Caverns, re-opened in 2003 and is accessible by a recently completed gondola tramway from town.

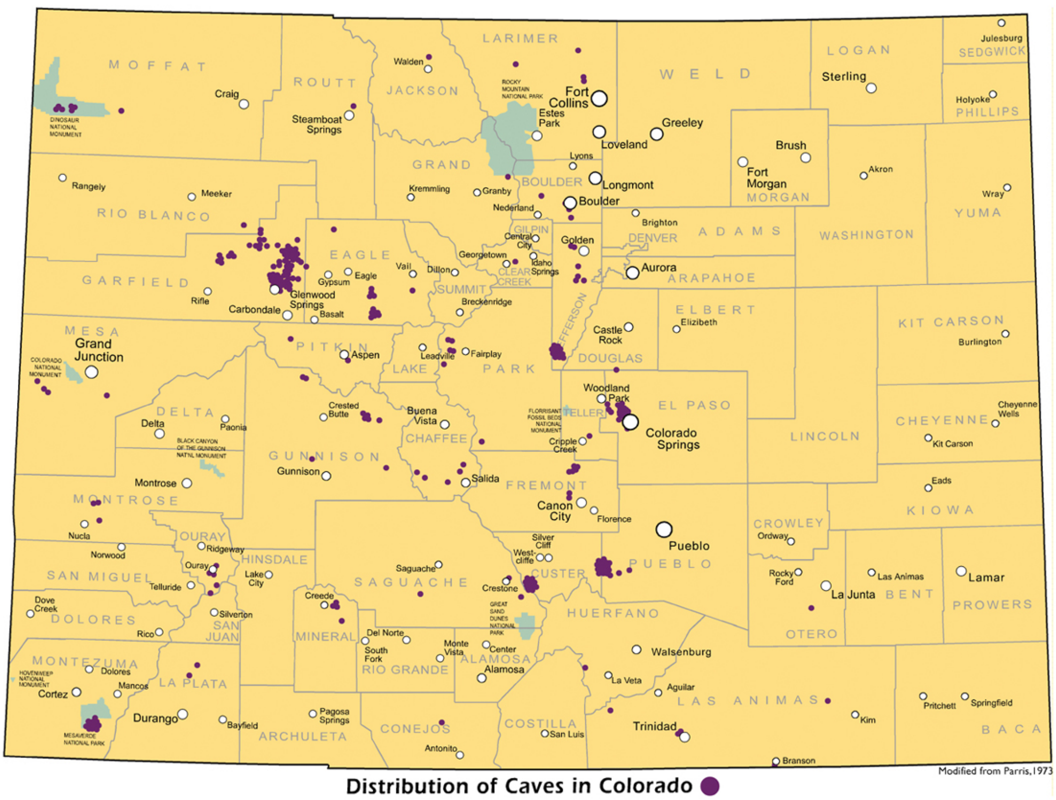

Figure 27. Distribution of caves in Colorado. From Matthews and others (in prep.); modified from Parris (1973).

Although the limestone caves are most impressive, there are many other types of caves in the state including ice caves at the base of glaciers and shelter caves, both of which are natural cavities large enough to permit human entry but not extending into total darkness. Shelter caves form when rocks in a cliffside erode more quickly than the overlying, resistant caprock. The best-known shelter caves are in Mesa Verde National Park. Clay caves form in riverbanks composed of weak shale, and are created by piping of the fine-grained sediment. More than 100 clay caves have recently been documented in the Mancos Shale between Montrose and Grand Junction. One of them is 2,000 ft (610 m) long. A cave along Highway 6 west of Golden is assumed to have opened by fault movement. Purgatory caves form in igneous or metamorphic rock where slot canyons are eroded and then covered by rocks falling into the crevasses. Eighteen “lost” caves are listed in old Colorado records, but have not been rediscovered.

Quaternary Volcanoes – The oldest known Quaternary volcanism in the state occurred about 1.5 million years ago in the Roaring Fork River Valley, about 10 mi (16 km) north-northwest of Aspen. There, magma issuing from the Crystal River fault produced a cinder cone and small basalt flows totaling about 500 ft (152 m) in maximum thickness. Just northeast of McCoy, volcanism produced two cinder cones and a basalt flow that has been radiometrically dated at approximately 640,000 years old. More-recent eruptions have been recorded near the junction of the Colorado and Eagle Rivers, where volcanic flows overlie modern topography. The most recent volcanism known in Colorado appears near the town of Dotsero (Figure 28). Using a charcoal sample recovered from a tree that had been buried by the falling ash, the Dotsero volcano and associated basalt flow has been radiometrically dated as 4,150 years old (Giegengack, 1962).

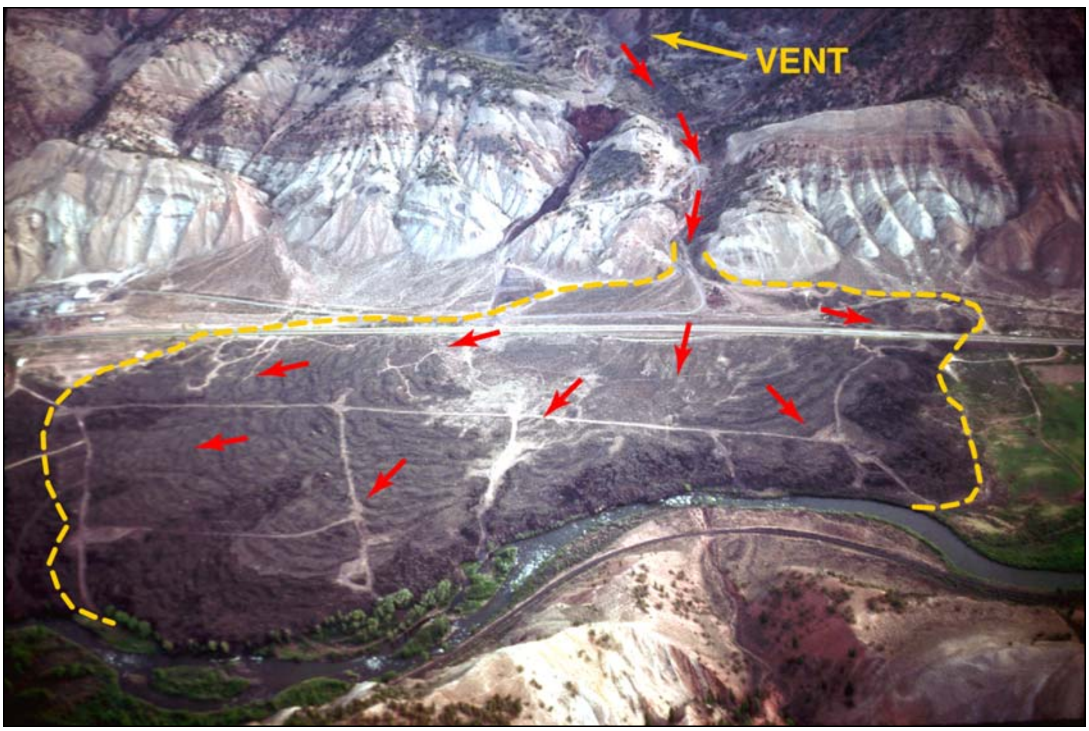

Figure 28. Aerial view of the Dotsero volcano and basalt flow, Colorado’s youngest. Photo by James Soule, Colorado Geological Survey.

Quaternary Faults – Faults have moved as a consequence of earthquake rupture and created offsets in Quaternary deposits in many basins in Colorado. According to a recent inventory by the Colorado Geological Survey (Widmann et al., 1998), there are at least 90 faults in the state that have moved during the Quaternary Period (Figure 29). Eight of these faults have had documented movement in the past 15,000 years. Many of these younger faults are associated with the Rio Grande Rift, running northward from New Mexico through the San Luis Valley to near Steamboat Springs. Another area having significant Quaternary faulting is in western Colorado, along the Uncompahgre Plateau and the Paradox and Big Gypsum Valleys south of Grand Junction.

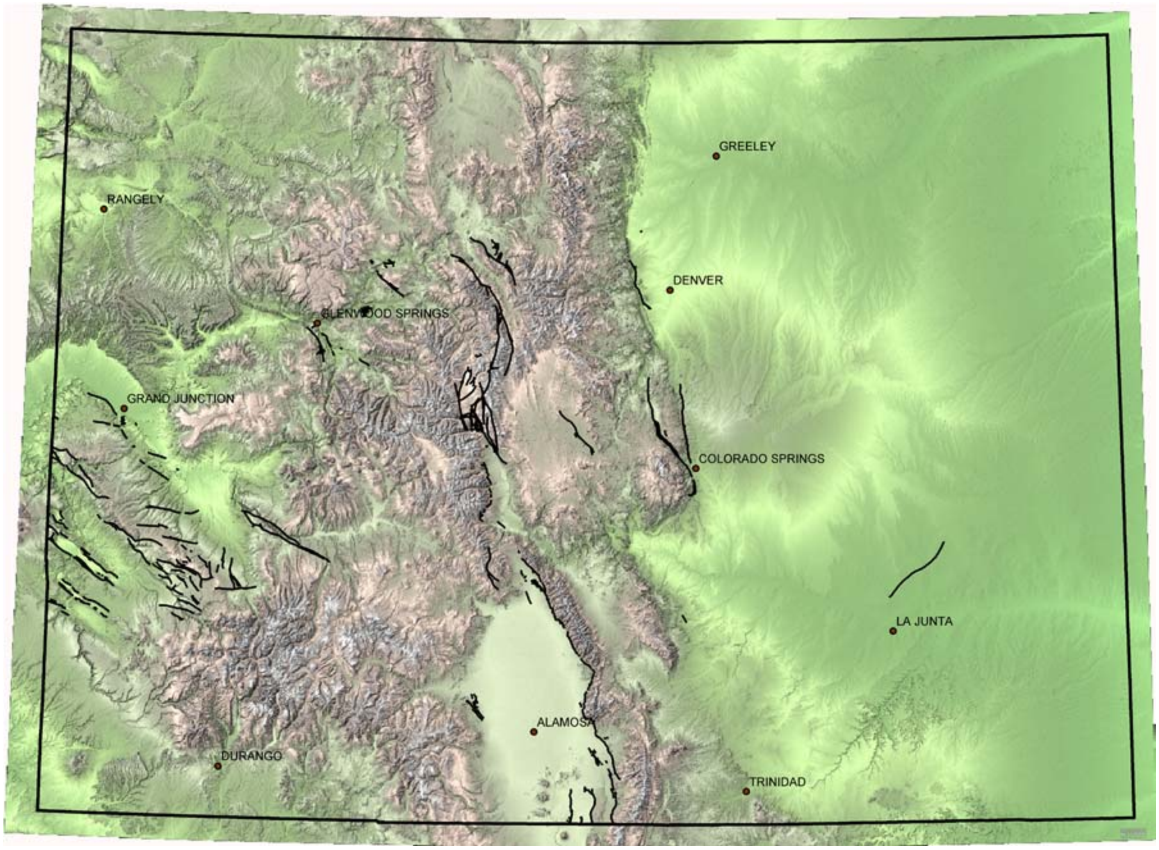

Figure 29. Distribution of Quaternary faults in Colorado. From Matthews and others (in prep.); modified from Widmann et al. (1998).

CONCLUSIONS

1) Colorado has three major physiographic provinces that roughly trend north-south through the state – the Great Plains, Southern Rocky Mountains, and Colorado Plateau provinces. Two other provinces – The Middle Rocky Mountains and the Wyoming Basin – occupy the far northwest corner of the state.

2) Colorado's location, far inland from any ocean, produces a semiarid climate with hot summers and cold winters. Its varied topography greatly influences statewide patterns of precipitation, temperature, and air movement. Except for the high mountains, most of the state has a deficit water balance as a result of low precipitation and high evapotranspiration rates.

3) Colorado contains an abundance of igneous, metamorphic, and sedimentary rock types, with rock formations representing every major geologic era and period, with the exception of the Silurian Period. The Cretaceous- to Tertiary-age structural deformation episodes associated with the Laramide uplift of the Rocky Mountains, and subsequent late Cenozoic volcanism, block faulting, and uplift have produced Colorado's varied and complex

geology and physiography. The resulting landscape consists of structural basins filled with Paleozoic, Mesozoic, and Cenozoic sedimentary rock formations, separated by structural uplifts cored by Precambrian crystalline rocks.

4) During the Quaternary Period, glaciers, water, wind, gravity, volcanic eruptions, and faulting modified the Tertiary landscape to produce the recent sediment deposits and landforms we see in Colorado today. Many of these geomorphic processes are active today.

5) Colorado's mineral and water resources, and its geologic hazards, are closely associated with the state's geography, climate, surface and subsurface geology, geomorphology, and past and present geologic processes. An understanding of the state's geology and geologic history – including the nature of its bedrock formations, Quaternary deposits, and geomorphic processes – is crucial in order to wisely and efficiently develop Colorado's mineral and water resources, and to protect the public from its many geologic hazards.

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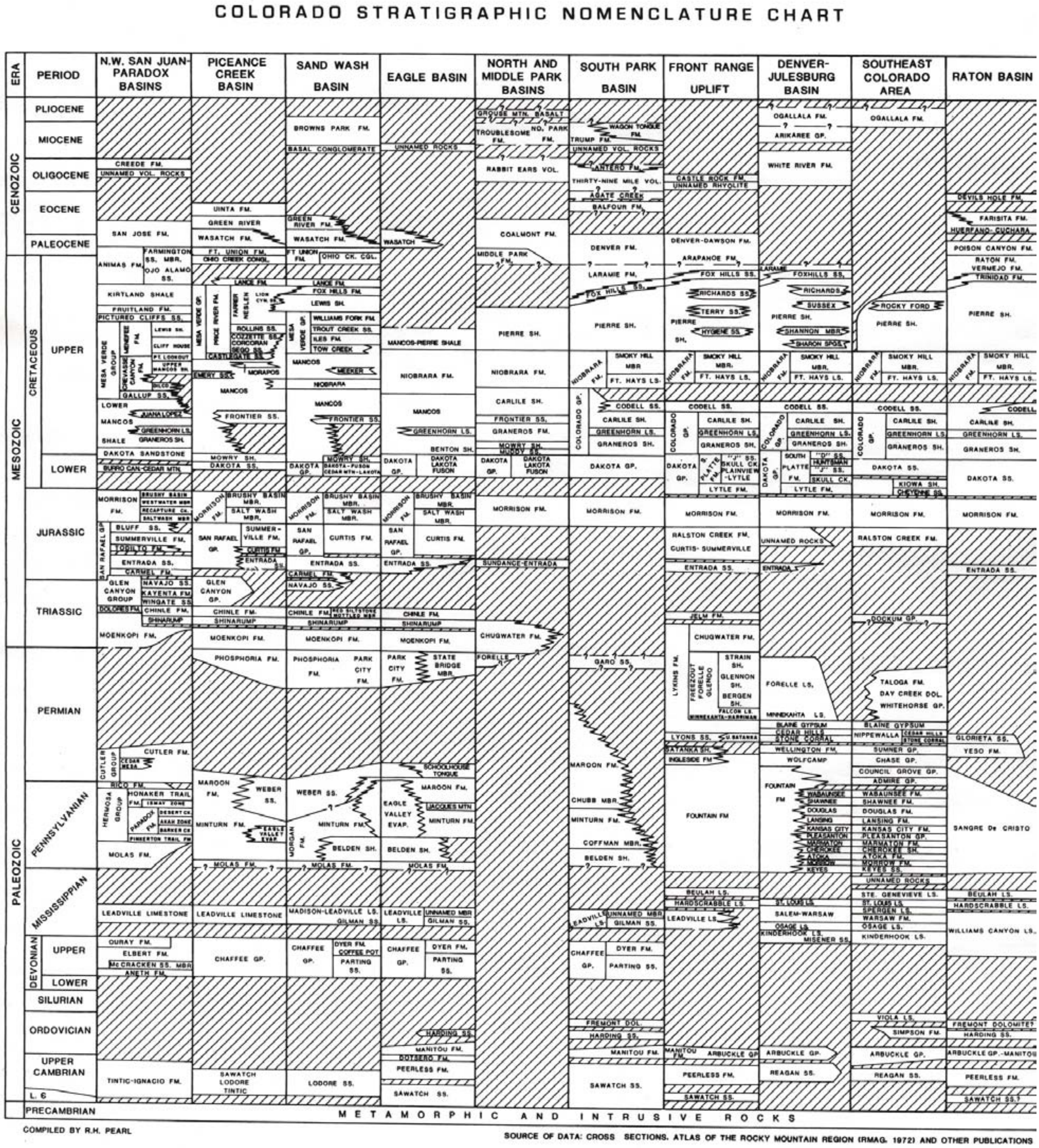
APPENDIX A. STRATIGRAPHIC NOMENCLATURE CHART OF COLORADO



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OVERVIEW OF THE GEOLOGY OF COLORADO PART II: MINERAL AND MINERAL FUEL RESOURCES, WATER RESOURCES AND WATER QUALITY, AND GEOLOGIC HAZARDS

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Key Terms: economic geology, mineral resources, water resources, ground water, water quality, geologic hazards

ABSTRACT

Colorado's mineral and water resources, and its geologic hazards, are closely associated with the state's geography, surface and subsurface geology, geomorphology, and past and present geologic processes. The state's mineral resources include precious and base metals, oil and gas, oil shale, coal and coalbed methane, radioactive minerals, industrial and construction minerals, and gemstones and ornamental stones. The state has locally significant surface- and ground-water resources. However, these resources are finite and are often located far away from the main population centers and agricultural users. Many of Colorado's western-slope streams are diverted to eastern-slope cities and agricultural areas via trans-mountain water diversion projects. The state's principal ground-water aquifers include Quaternary-age alluvial aquifers, basin-fill sediments, sedimentary rock aquifers, and volcanic and crystalline rock aquifers. The natural interaction between water and rock in certain parts of Colorado can produce poor water quality, in the form of dissolved metals and acid rock drainage, salts and selenium, and radioactive elements. Geologic hazards and their associated planning and mitigation issues are becoming increasingly critical as Colorado's population continues to grow. Nearly all of the state's populated areas are subject to geologic hazards or constraints of one kind or another. These include expansive soil and bedrock, collapsible soil, evaporite and limestone karst, coal mine subsidence, coal-seam fires, landslides, rockfall, debris flows, stream floods, avalanches, earthquakes, radon gas, and environmental hazards. Most of these phenomena are natural geologic processes that become hazardous when human land uses take place in susceptible areas. In some cases, these human uses can trigger, reactivate, or increase the risks from a geologic hazard. The state has several land-use laws designed to facilitate the recognition of and planning for geologic hazards, in order to reduce these potential impacts. An understanding of geology is crucial in order to wisely and efficiently develop Colorado's mineral and water resources, and to protect the public from geology-related threats to life, health, and safety.

INTRODUCTION

Colorado has a rich history of varied land uses, including mining and agriculture, which have relied upon the state's natural resources. The state contains many mineral and mineral fuel resources, which are found in a wide variety of geologic settings. In addition, its rivers, lakes and reservoirs, and aquifers are important water resources that are used by farmers, ranchers, industry, and urban populations. The quality of Colorado's water sources is very much influenced by the local geology in some areas. Likewise, the hydrologic makeup of the state's numerous geothermal hot springs and wetlands are influenced by the surrounding geology. The characterization and prudent development of all of these limited natural resources depends to a large degree on an understanding of geology and geologic processes.

Colorado's topography and climate are conducive to geologic processes that impose a variety of hazards and constraints on human uses. In the mountains, plateaus, and foothills, gravity-driven hazards such as rockfall and landslides abound. Seasonal thunderstorms can produce debris flows from the hillsides and torrential flash flooding in the stream valleys. Even the flatter areas along the valleys and plains are at risk, as they are often underlain by soil or rock that can swell, settle, or dissolve. Often, these hazards become more pronounced as a consequence of human activities. This may involve physical modification (such as cutting the toe of a landslide or artificially routing a channel across a debris-flow fan), or it may involve increasing the amount of moisture in the ground (from using irrigation or other means), both of which may destabilize the underlying soil or rock.

A booming population made Colorado the third-fastest-growing state in the nation during the 1990s. Access to and development of its mineral, mineral fuel, and water resources have become increasing critical issues, especially in light of conflicting demands in public consumption and land uses. As Colorado grows in population, the populace is increasingly exposed to geologic hazards, which must be understood and properly avoided or mitigated in order to reduce threats to life and property.

This paper, which is the second (Part II) of a two-part overview paper, includes abbreviated descriptions of Colorado's geology-related natural resources and hazards, many of which will be described in greater detail in other individual chapters and technical papers of this volume. A discussion of the state's physiographic, climatic, and geologic settings, and geologic history was presented in the first paper (Part I) from this overview (Noe et al., 2003).

MINERAL AND MINERAL FUEL RESOURCES

Colorado has a great variety of mineral resources that society has found useful or necessary for survival. The mountainous areas of the state have yielded precious metals such as gold, silver, lead, zinc, molybdenum, copper, and tungsten. More than 770 different minerals have been catalogued in the state (Eckel, 1997). Energy resources of oil, natural gas, oil shale, uranium, and coal are abundant. Industrial minerals and construction materials abound as well. The economic importance of Colorado's mineral and mineral fuel resources continues today. The total value of production in 1999 was 3.8 billion dollars. Unless otherwise noted, the economic and production values used this section are from Wray and others (2001).

Precious and Base Metals

Most of the significant metal deposits in Colorado are located within the Colorado Mineral Belt, a northeast-southwest trending zone that extends from the La Plata Mountains near Durango to the Front Range just north of Boulder (Figure1). The famous mining districts and towns of Telluride, Crested Butte, Silverton, Aspen, Leadville, Fairplay, Breckenridge, Georgetown, Central City, and Jamestown are located in this relatively narrow (10 to 60 mi wide or 16 to 97 km wide) but productive belt. The precious-metal (gold and silver) and base-metal (lead, zinc, and copper) deposits of the mineral belt occur in veins that were created by hydrothermal

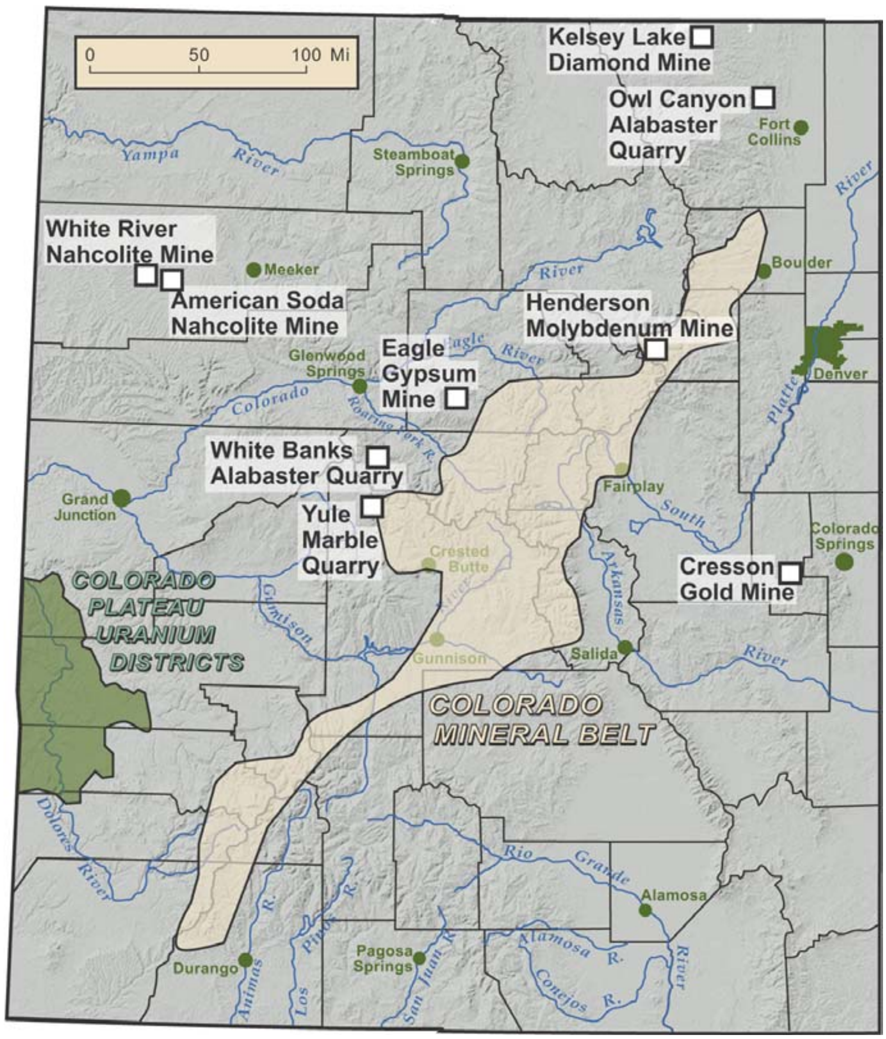

Figure 1. The Colorado Mineral Belt and active mines in Colorado. From Matthews and others (in prep.); modified from Wray et al. (2001).

solutions associated with igneous intrusions and volcanic eruptions, most of which occurred during the Laramide mountain-building event between 75 and 42 million years ago. Although precious and base metals are the main commodities mined from the Colorado Mineral Belt, other important metals have also been extracted. For example, in the early 20th century around the time of World War I, the Boulder area produced more tungsten than any other region in the U.S.

At various times throughout its history, Colorado has been the leading U.S. producer of gold, silver, molybdenum, lead, zinc, uranium, and tungsten. In 2000, it ranked second among the states in molybdenum production, behind Arizona, and seventh gold production. Other metals that have been mined in Colorado include copper, tin, vanadium, iron, beryllium, lithium, rare earth elements, thorium, tantalum, and manganese.

Gold – The early history of the State of Colorado is directly tied to the first documented discovery of placer gold at Auraria (at what is now downtown Denver) in the summer of 1858, from stream gravels near the confluence of Cherry Creek and the South Platte River. This discovery led to the first Colorado gold rush. The Denver-area gold placers proved to be small and were quickly depleted. However, prospectors subsequently discovered rich vein and placer gold deposits at Idaho Springs and Central City. By the time Colorado became a state in 1876, many mining districts and new cities had sprung up throughout the area.

Large placer gold mines have operated in several places in Colorado. The largest of these were near Fairplay along the upper South Platte River, and near Breckenridge along the Blue River and French Gulch. Dredges equipped with huge buckets scooped gravel out of the riverbeds and processed it to concentrate the gold. In some places, hydraulic mining methods were used. Large piles of processed-gravel tailings still line these rivers years after the mining has ceased. Today, some of the sand and gravel pits along the South Platte River and Clear Creek, near Denver, produce small amounts of gold as a byproduct of their aggregate operations. There are many places in Colorado would-be prospectors can try their hands at panning for gold.

Some of Colorado's most significant metal deposits are not associated with the Colorado Mineral Belt. The largest single gold mining district in Colorado, by far, is Cripple Creek, which lies well south of the belt. Over 21 million oz (595 metric tons), nearly one half of all gold mined in the state, have been produced in the district since its discovery in 1891. The Cresson Mine is the only active gold mine in Colorado, and produced over 230,000 oz (0.65 metric tons) of gold in 1999. The gold deposits are related to an Oligocene-age (28-32 million years ago) intrusive center that may be the subsurface portion of an eroded volcano.

Silver, Lead, and Zinc – The enormous lead-zinc-silver deposits at Leadville, Gilman, and Garfield-Monarch were formed as replacement deposits in carbonate sedimentary rocks, particularly the Mississippian Leadville Limestone. These deposits were formed by hydrothermal solutions, which contained dissolved metals, flowing and ascending along faults and fractures. When the solution reached a receptive dolomite or limestone bed capped by an impermeable layer, such as an igneous sill or a shale layer, it spread through the receptive unit, removing calcium, magnesium, and carbon dioxide and replacing it with base-metal sulfides and other minerals.

Molybdenum – The Henderson Mine, near Empire, is the largest operating primary producer of molybdenum in the world. At Henderson, molybdenite occurs in quartz veins and veinlets within a Tertiary-age, granitic intrusion. The ore is mined more than 3,000 ft (914 m) below the surface, and then transported by an underground conveyor belt for 10.5 mi (17 km) under the Continental Divide to the ore-processing facilities in Summit County. The geologically similar Climax molybdenum deposit, northeast of Leadville, is now inactive, but it once was the world's largest provider of the metal. The Mount Emmons molybdenum deposit, west of Crested Butte, is also a world-class resource, but has not yet been developed.

Titanium – The world's largest, single identified resource of titanium metal is located in Gunnison County. The Powderhorn titanium deposit, consisting of the minerals ilmenite and perovskite, has not yet been mined, but recently underwent an evaluation by a major mining company to determine the economic feasibility of a new mining operation there. Although the geological resource is very large, the extraction of titanium from its host rock is very expensive and energy intensive. For a more complete discussion about Colorado's titanium and other strategic-mineral deposits, see Schwochow and Hornbaker (1985).

Precambrian-Age Mineral Deposits – Precious metals, base metals, and some tungsten have been mined in Colorado from small deposits formed in late Precambrian (Proterozoic) time. These deposits are scattered throughout the mountain region but are especially numerous in Gunnison, Saguache, Fremont, and Chaffee counties of south-central Colorado. The deposits tend to be smaller than the Laramide-age deposits. Metamorphism and tectonic movement subsequent to the formation of these ancient deposits has generally made them more difficult to understand – and to mine – than the younger deposits. The Sedalia Mine near the town of Salida was the state's largest producer of copper in the late 19th and early 20th centuries.

Pegmatites

Vein-like bodies of pegmatite occur in several parts of Colorado's mountains, in or near large granite stocks and batholiths. These are all Precambrian in age. Large crystals of feldspar, quartz, and mica are the dominant minerals, but many of these pegmatites also contain variable quantities of rare and sometimes valuable minerals. Colorado pegmatites yield beryllium, lithium, niobium, tantalum, thorium, yttrium, cerium, and lanthanum, all of which have high-tech applications. Larger deposits in other areas of the world currently meet the demand for these metals, however, and pegmatites are currently not being mined in Colorado.

Mineral Fuels

Oil and Natural Gas – Historically, 36 of Colorado's 64 counties have produced oil, and 39 counties have produced natural gas. Cumulative production from the nearly 1,400 fields that have been discovered in Colorado now stands at about 1.83 billion barrels of oil and 10 trillion cubic feet of gas. In 1999, Colorado ranked tenth in the United States in daily oil production with 54,000 barrels of crude oil per day, and sixth in daily gas production with 1.92 billion cubic feet per day. In recent years, natural gas production has been on the increase in the state while crude oil production has declined. Colorado's four, principal oil and gas producing regions are the Sand Wash Basin and Piceance Basin region, the Paradox Basin and San Juan Basin region, the Raton Basin region, and the Denver Basin region (Figure 2).

In 1876, the second oil field in the United States was established near Florence, Colorado. Oil was found in fractures and fissures in the Cretaceous Pierre Shale. The first gas production was established in the northwestern part of the state in 1890, in the Piceance Basin. Northwestern Colorado is also the home of Rangely Oil Field, the largest oil field in the Rocky Mountain region. Oil production was first established on this huge anticline in 1902, from shallow reservoirs of fractured, Cretaceous Mancos Shale. Much-larger production was established in

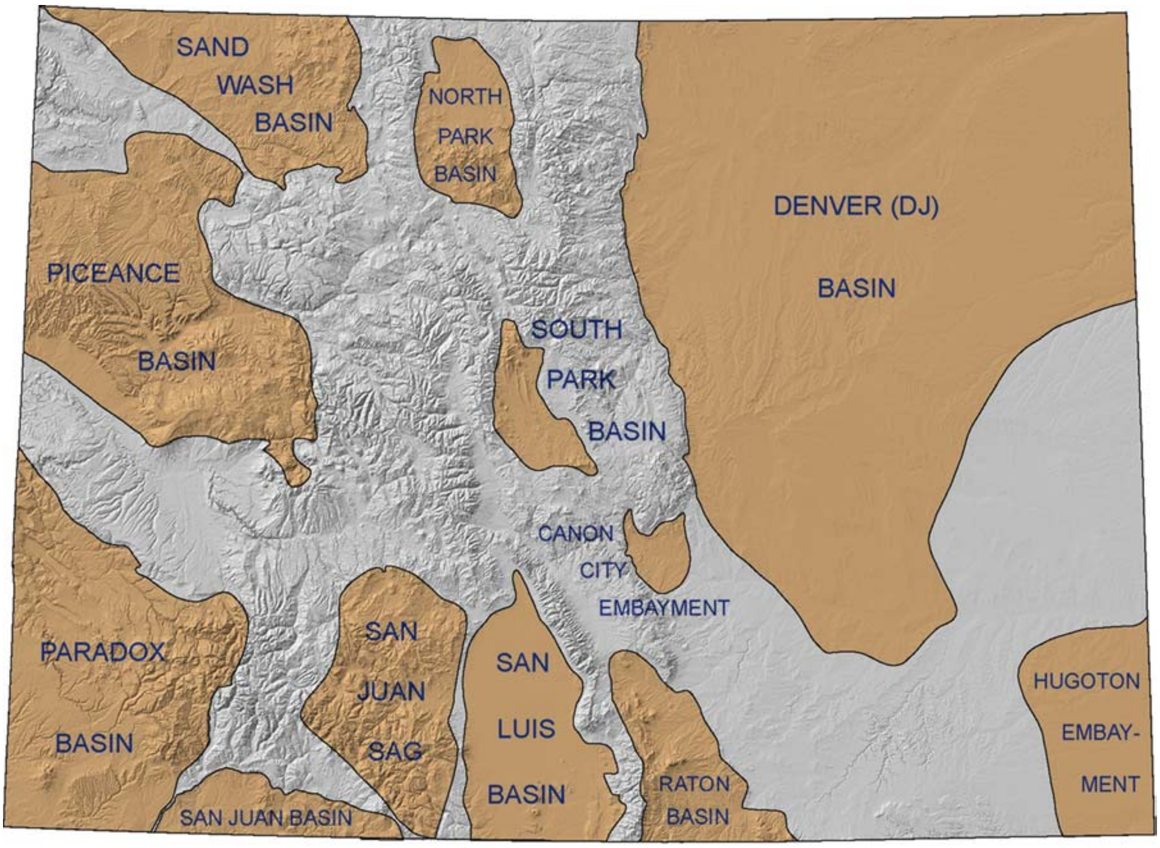

Figure 2. Oil and gas producing basins in Colorado. From Matthews and others (in prep.); modified from Wray et al. (2001).

1933 from the deeper, Permian Weber Sandstone reservoir (Bass, 1964). Cumulative production from Rangely Field at the end of 1999 was 845 million barrels of oil and 761 billion cubic feet of gas. Today it is nearing depletion.

Oil Shale – There are enormous deposits of oil shale (i.e., kerogen-bearing marlstone) in the Tertiary Green River Formation, in Western Colorado. It is estimated that these reserves might produce 600 to 800 billion barrels of oil. In order to yield oil, the marlstone must be mined, crushed, and retorted at 900°F (482°C), or processed *in situ*.

Despite a century of interest, energy companies have not yet developed cost-effective methods of retrieving the oil, and the feasibility of developing this resource in the future is uncertain. In addition to the mining and processing technical problems, there are major obstacles relating to the amount of water needed and the amount of waste produced. However, as world supplies of conventional oil near their peak production, major companies are once again exploring the economics of producing this resource. The Shell Oil Company is currently evaluating an *in-situ* process at a location in the Piceance Basin.

Coal and Coalbed Methane – Coal is one of Colorado's most widespread mineral and energy resources, underlying 29,600 square miles or 28 percent of land in the state. The majority of coal was formed approximately 80 to 65 million years ago in the swamps of the late Cretaceous and early Tertiary periods. Coal has been commercially mined for at least 140 years in the state. Some of the earliest coal mining began in the 1860s in the Denver Basin, to the north and west of Denver. Early coal mining occurred around the turn of the 20th century in Colorado Springs and in the Raton Basin, west of Trinidad. Today, most of the coal production comes from surface mining in the northwest part of the state (Figure 3).

Figure 3. Mining coal with large augers in a surface mine near Hayden. Photo by Christopher Carroll, Colorado Geological Survey.

Colorado coal is especially valuable because of its relatively low content of sulfur, ash, and mercury. Approximately 45 percent of the state's coal production is burned in Colorado power plants, while 49 percent is shipped to other states where clean, but more expensive, Colorado coal is mixed with the sulfur-laden, local coal. The Energy Information Administration (EIA) estimates that Colorado has approximately 16.8 billion tons of coal reserves. The state is well situated to continue to provide high-quality, clean-burning coal to its citizens as well as to users in other states and countries.

Coalbed methane comes from biologic decay of peat and is generated during burial and heating of organic material as it converts into bituminous coal. This colorless, odorless, highly explosive gas, which has been a long-time menace to coal miners throughout the centuries, has become a valuable source of energy. Today, the San Juan Basin of Colorado and New Mexico is the most prolific coalbed methane-producing basin in the world. Coalbed methane accounted for 54.5 percent of Colorado's total gas production in 1998. These productive coalbed methane areas are in only seven of the state's 64 counties. That number is bound to increase in the future as technological advances and geologic understanding of coalbed methane continues.

Uranium and Vanadium – The 1950s witnessed one of the most intensive treasure hunts in the state's history, as thousands of fortune hunters swarmed over every inch of the plateaus and canyon lands of western Colorado searching for radioactive deposits of uranium and vanadium. The French physicists, Marie and Pierre Curie, for their Nobel Prize-winning research on radioactive substances, processed uranium ore from Colorado for its radium content.

Typically, these deposits occur in the Jurassic Morrison Formation, and most commonly in the Uravan mineral belt, part of the Colorado Plateau uranium districts . They were created when ground water carrying dissolved uranium and vanadium flowed through the porous sandstone and encountered carbon-rich, fossilized plant material. The fossilized plants acted as chemical traps, precipitating the metals out of solution. Occasionally, fossil tree trunks composed almost entirely of uranium minerals, have been found. The largest uranium mine in Colorado was the Schwartzwalder mine, located between Boulder and Golden, which closed in 2000. Uranium mineralization at this mine occurred in veins in Precambrian metamorphic rock.

Industrial Minerals and Construction Minerals

Colorado has vast resources of many types of nonmetallic, industrial minerals. Fueled by the state's tremendous rate of growth over the past decade, construction minerals are by far the most economically significant of the industrial mineral resources in the state.

Aggregate – Of the construction minerals, aggregate is the most commonly used commodity. Aggregate includes sand, gravel, and crushed rock – the largest components of concrete, asphalt, and road base. It is estimated that each person in the Front Range uses about 14 tons of aggregate per year. Only production of natural gas, oil and coal surpasses the value of Colorado aggregate production. As the third-fastest growing state, Colorado ranked seventh in the nation in 2000 for sand and gravel production, with a little over 50 million tons.

Sand and gravel are produced statewide (Figure 4) from various types of Quaternary deposits, including alluvial floodplains and terraces and glacial, eolian, and colluvial deposits. Some of the best quality gravels are found in the modern floodplains of major rivers, such as the South Platte and Cache La Poudre Rivers and Clear Creek along the Front Range. Although quality is important, the distance of transporting sand and gravel to construction projects is the single, most important factor influencing the price. It is therefore crucial to locate high-quality deposits near the places where construction is occurring.

As the state's sand and gravel deposits are either depleted or rendered inaccessible for future use by development, the demand for crushed rock for use as road base and in asphalt is increasing. Production of crushed rock increased by 50% from 1995 to 2000. Fortunately, there is a large supply of granite and gneiss in Colorado's mountainous regions. Quartzite and limestone are also good sources of crushed rock and are quarried in some locations; however, the deposits are not as large or widely available.

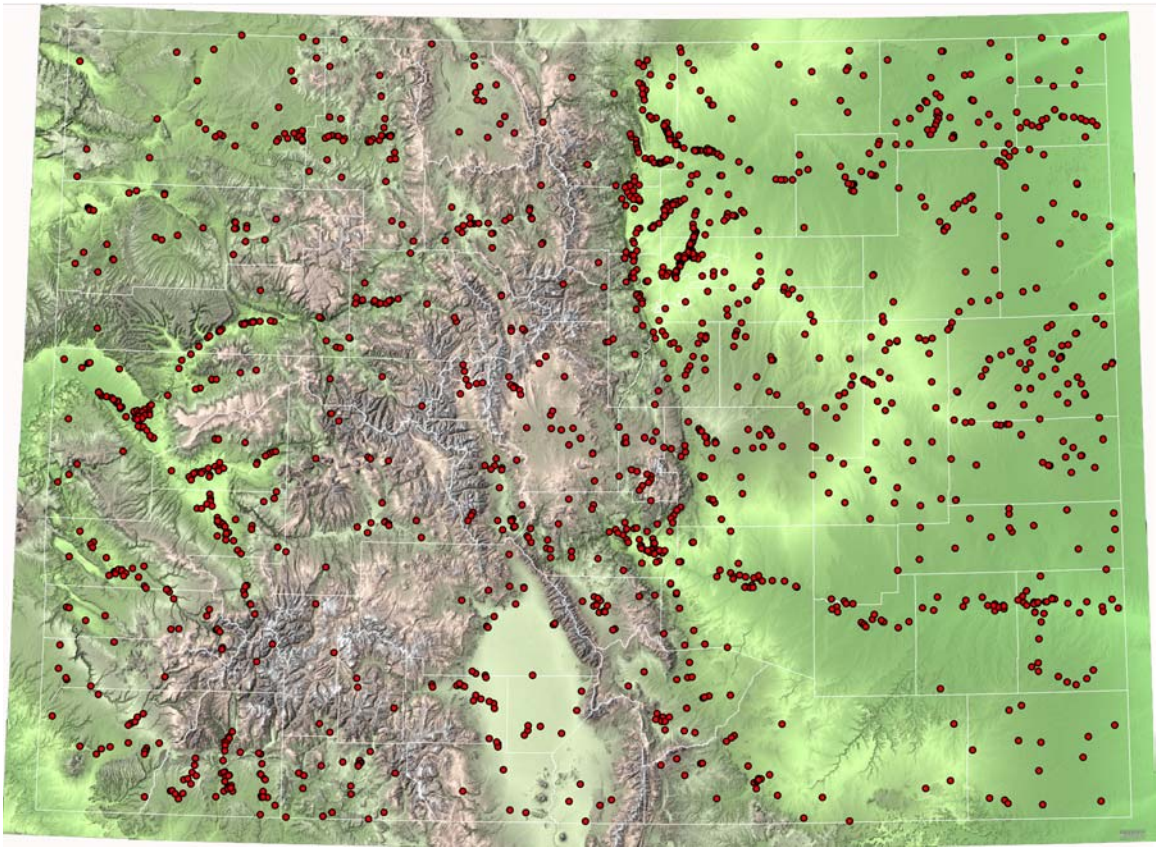

Figure 4. Active permitted mines in Colorado, most of which are sand and gravel pits along stream valleys or mountainside crushed-rock aggregate quarries. From Matthews and others (in prep.).

Limestone – Most of the industrial-grade limestone and dolomite currently being mined in Colorado is from the Cretaceous strata along the Front Range, especially the Fort Hayes Limestone Member of the Niobrara Formation at La Porte and near Portland. Limestone and dolomite are widely used by the construction industry. Crushed limestone and dolomite are used as road base or decorative stone, and large blocks of limestone and dolomite can be used as building stone. Lime is also the key ingredient in cement, and is frequently used in agriculture as a soil additive to help break up clay soils, neutralize acidic soils, and as an essential plant food. Limestone has also been quarried for use in the sugar refineries of the eastern plains.

Clay – There are hundreds of uses for clay, but in Colorado it is used primarily to make bricks. The Cretaceous formations are major sources of clay. Along the Front Range, numerous clay pits have been dug along steeply dipping strata in the Dakota Sandstone and the Laramie Formation. Another area of clay mining near Denver is in Douglas County, where pits have been dug in a paleosol and other clay-bearing layers in the Cretaceous-Tertiary Dawson Formation. At Rocky Flats, between Golden and Boulder, the Pierre Shale is mined and thermally processed into lightweight aggregate. The Coors Porcelain Company has mined clay for many years for use in pottery and low-temperature ceramic ware.

Dimension Stone – In Colorado, a variety of local rock types including marble, sandstone, granite, rhyolite, and travertine are quarried and used for dimension stone. Perhaps the most famous is the Yule Marble, near Marble, which was formed when the Mississippian Leadville Limestone was subjected to contact metamorphism by an intrusive Tertiary stock. This pure, white marble was used in the Tomb of the Unknowns in Arlington National Cemetery and the Lincoln Memorial in Washington, D.C. The Permian Lyons Sandstone of Boulder and Larimer counties has been used as dimension stone since the mid-to-late 1800s, and is used extensively in buildings on the University of Colorado's Boulder campus.

The Aberdeen Granite of Gunnison County is one of the most widely used granites in the state; the state capitol building and the steps leading up to its entrances contain 283,500 ft^3 (8,028 m^3) of this stone. The colorful interior walls of the capitol are made of limestone (called "Beulah Marble") that was quarried west of Pueblo. Cotopaxi Granite of Fremont County was used in the construction of the base of Denver City Hall. Pikes Peak Granite of Douglas County and Silver Plume Granite of Clear Creek County are other notable sources of dimension stone.

The light gray to pinkish, Wall Mountain Tuff of Douglas County, known in the building industry as the Castle Rock Rhyolite, has been widely used as building stone in the Front Range. Denver's Trinity United Methodist Church was built using this rock.

The most significant travertine deposit in Colorado is located near Wellesville. Denver General Hospital, the Gates Rubber Company, and the Bus Terminal Building are some of the buildings in the Denver Metro area constructed of Colorado travertine (Murphy, 1995).

Evaporite Minerals – Gypsum, salt, and nahcolite are industrial minerals with a common origin as evaporites deposited in Permian-Pennsylvanian inland seas. The most widespread deposits, in the Paradox Valley in southwestern Colorado, have not been extensively mined because they are located far from major population centers and transportation routes. In contrast, gypsum deposits in the Eagle Basin near the town of Gypsum are mined to make drywall (Figure 5). This mine and its drywall plant are located adjacent to Interstate 70 and only a few miles (a few km) from a rail line, which makes it economically viable.

In the Piceance Basin of northwestern Colorado, nahcolite (sodium bicarbonate) is mined by circulating hot water down into the deposit. The dissolved bicarbonate is pumped back to the surface and manufactured into baking soda, most of which is exported. Construction of a plant for converting the sodium bicarbonate to sodium carbonate (soda ash) began in 2000. Soda ash is widely used in industry as an ingredient in chemical manufacturing, in glass and fiber optics. It is used to remove sulfur dioxide – a major pollutant – from the flue gas of coal-fired power plants. This deposit may contain 30 billion tons of nahcolite, the largest in the world.

Gemstones and Ornamental Stones

More than thirty different varieties of gems and ornamental stones are known to occur in Colorado. The most famous and historically valuable gems are diamonds. In 1975 diamonds were discovered in Colorado. The Kelsey Lake diamond mine, near the Wyoming border north of Fort Collins, began producing diamonds on a commercial scale in 1996 – the only commercial

Figure 5. A gypsum quarry in the Pennsylvanian Eagle Valley Evaporite, near Gypsum. Photo by David Noe and Jonathan White, Colorado Geological Survey.

producer in the United States. In 1997, the mine produced a yellow stone weighing 28.3 carats, the fifth-largest diamond ever found in the United States. Weighing in at 16.87 carats after cutting and polishing, it became the largest faceted diamond ever produced in the U.S.

The State Mineral is rhodochrosite, a red manganese carbonate mineral found in eighteen of Colorado's counties. Rhodochrosite crystals from the Sweet Home Mine, near the town of Alma in Park County, are prized all over the world for their exceptional size, color, and quality (Figure 6). The finest specimens command prices of up to $100,000. Aquamarine, the State Gem, is a clear, blue variety of beryl. Mount Antero and nearby Mount White, in Chaffee County, are two of the best places in the world to collect specimens of this beautiful mineral.

Other notable gem-quality minerals that have been found in Colorado include amazonite, garnet, topaz, tourmaline, lapis lazuli, quartz crystal, smoky and rose quartz, amethyst, peridot, sapphire, turquoise, and zircon. Agate, chalcedony, and jasper are found in many places. Alabaster, a fine-grained, compact variety of gypsum used to make elegant vases and other decorative items, is quarried in the foothills northwest of Fort Collins and at a new quarry south of Carbondale.

Figure 6. Large crystals of rhodochrosite surrounded by tetrahedrite and quartz. This specimen was extracted from the Sweet Home Mine, near Alma. Specimen courtesy of Dave Bunk Minerals; photo by Jeff Schovil.

WATER RESOURCES

Historically, development throughout the semi-arid West has depended upon the availability of water resources. This is underscored by the drought of 2001-2002, in which the state was beset by low snowfall and rainfall and overall dry, warm conditions. The period of September 1, 2001 to August 30, 2002, was the driest at most climate-observing sites since records have been kept (Pielke, 2003). Colorado's history of development is punctuated by societal reactions to wet and dry precipitation cycles. Estimations of water supplies and projections of water usage can be abruptly upset by drought cycles. In good times and economic prosperity, people often forget the lessons of the past and do not consider the limitations of the state's water resources during the inevitable dry periods.

Today, demands on Colorado's water supply include domestic, industrial, agricultural, wildlife, and recreational uses. With its ready access and storage capability, surface water has historically been and continues to provide the bulk of the state's water supply. Over-appropriation of this resource, however, combined with rapid urban growth and a lack of suitable and approved future storage reservoir sites, has focused attention on ground-water resources.

Several of the topics outlined in this section – water resources, aquifers, and quality, storage facilities, and hydrogeologic constraints on engineered projects – are the subjects of several papers in the “Dams/Water Resources,” “Tunnels/Underground Construction,” and “Ground Water/Environmental” chapters of this volume.

Ground-Water Resources and Aquifers

The use of ground water in Colorado for public supply and for domestic and industrial purposes dates back to before 1900. Ground-water resources currently supply approximately 18 percent of the state’s needs, about 2,300 million gallons per day (0.87 million m^3 per day), and ground-water development is continuing at a fast pace. With finite subsurface configurations of the state’s aquifers, and low recharge rates that are characteristic of a semi-arid climate, this resource should be considered finite with definitive limits and boundaries.

The occurrence and distribution of Colorado’s ground-water resource is physically linked to the state’s geography and underlying geology. The principal aquifers are categorized into four main types: 1) Quaternary-age alluvial aquifers associated with all of the state’s major river systems, 2) sedimentary rock aquifers such as those in the Denver Basin, 3) basin-fill sediments such as those in the San Luis Valley, and 4) volcanic and crystalline rock aquifers that dominate the mountainous regions.

Alluvial Aquifers – Alluvial deposits associated with major river systems are important and often prolific, shallow aquifers in Colorado. They consist of silt, sand, and gravel that have been deposited during recent geologic time by streams as sorted or semi-sorted sediment. On a regional scale, the principle alluvial aquifers are associated with ten of Colorado’s major river watersheds: the South Platte, Republican, Arkansas, Colorado, Yampa, White, Gunnison, San Juan, and Dolores Rivers, and the Rio Grande (Figure 7).

Sedimentary Rock Aquifers – The major sedimentary rock aquifers in Colorado consist predominantly of sandstones and limestones of varying ages. Many of these aquifers are located in structural basins that contain multiple geologic units or aquifers. Basin or statewide aquifer systems of this type are found in the Denver, Sand Wash, Piceance, Eagle, Paradox, San Juan, and Raton Basins, and the Dakota-Cheyenne Group and the High Plains aquifer (Figure 8).

Basin-Fill Aquifers – In addition to isolated sedimentary bedrock aquifers in the intermontane parks, Colorado’s mountainous regions contain basin-fill aquifers composed of erosional clastic material, and fractured crystalline (igneous and metamorphic) and volcanic rock aquifers. The intermontane basins of central Colorado contain a network of hydraulically interconnected aquifers within basin-fill deposits in the San Luis Valley, Wet Mountain Valley and Huerfano Park, and North, Middle, and South Parks. These unlithified to poorly lithified aquifers consist of sediments that were deposited by wind, water, and gravity, such as landslides and debris flows from erosion of the surrounding mountain ranges.

Volcanic and Crystalline Rock Aquifers – Precambrian granites and gneisses and geologically recent volcanic and igneous intrusive rocks represent the fractured, crystalline-rock aquifers that supply much of the domestic needs in the mountainous portion of the state. These aquifers may typically have low storage capacities. In some places, such as the rapidly growing mountain communities to the west of Denver, both the water supply and water quality from fractured

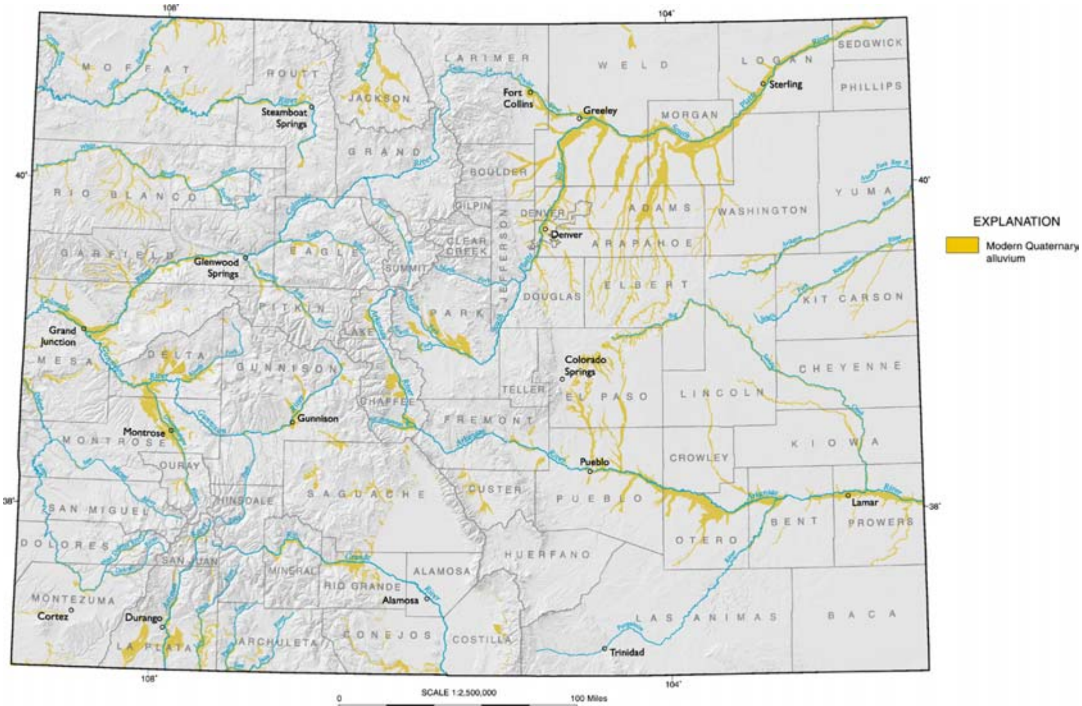

Figure 7. Distribution of alluvial aquifers in Colorado. From Topper and others (2003); modified from Tweto (1979).

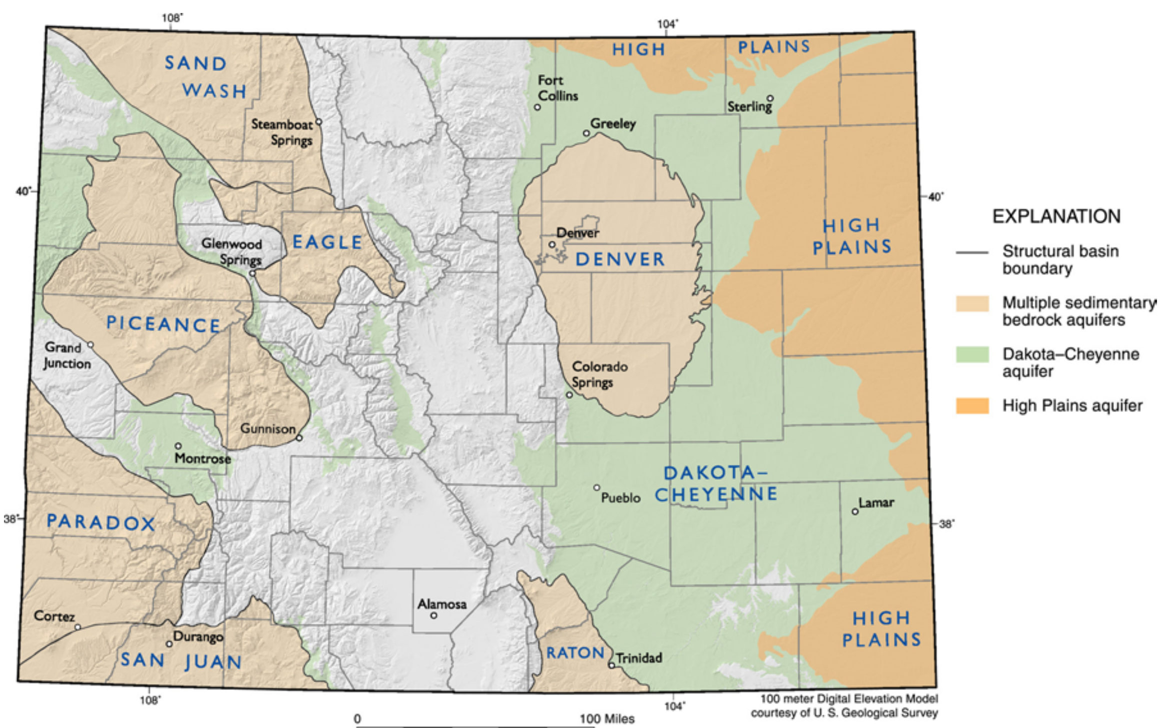

Figure 8. Distribution of sedimentary aquifers and related structural basins in Colorado. From Topper and others (2003).

crystalline-rock aquifers are becoming issues of concern. The state's principal mountain-region aquifers are shown in Figure 9.

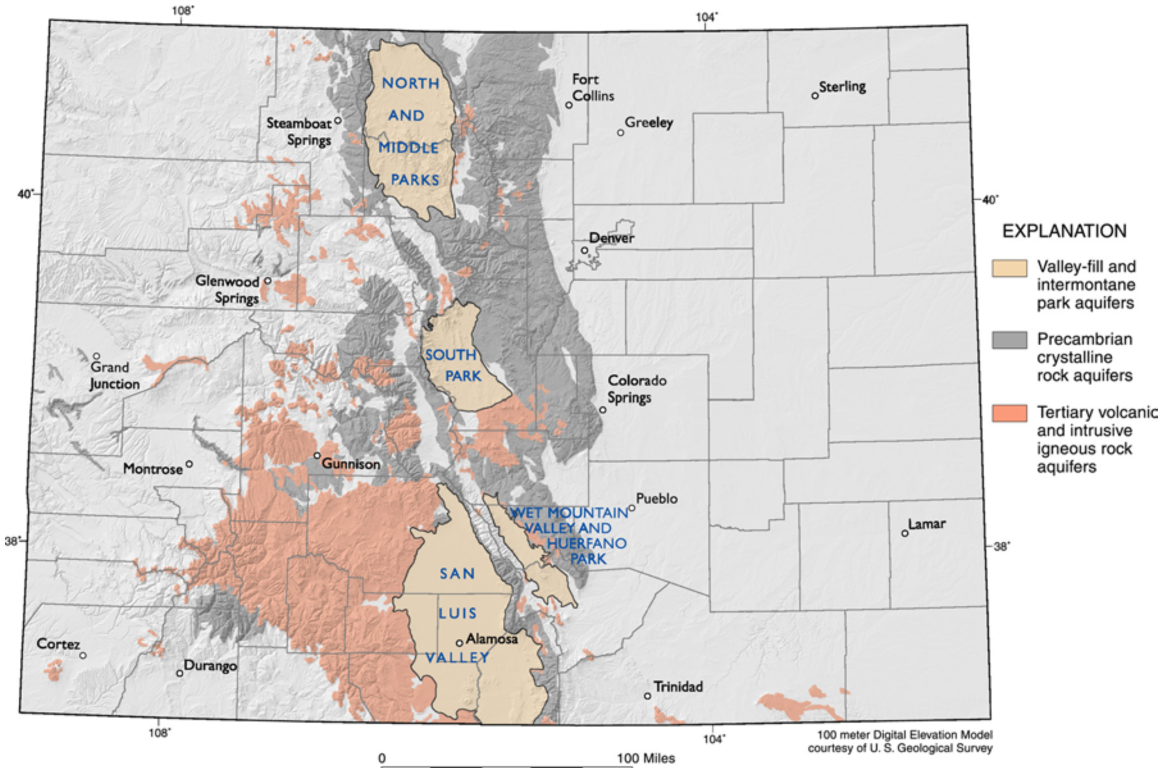

Figure 9. Distribution of mountain-region aquifers in Colorado. From Topper and others (2003).

Surface-Water Storage and Diversion

Colorado is a headwaters state: the state's principal streams begin in its mountains, plateaus, or plains areas, and its major rivers flow out of the state in all directions. Ninety percent of the available surface water is located in the western half of the state. Unfortunately, demands for that resource are highest in the eastern part of the state, where eighty-five percent of the population resides.

The major problem involving surface water in Colorado is how to move it from areas where it is abundant and under-utilized to areas where demand has outpaced supply. Complicated irrigation canals and water-supply delivery systems were developed soon after the state began to be settled. To keep up with demand in the eastern part of the state, large amounts of water are diverted from the western slope to eastern flowing streams through ditches and tunnels. There are thirty-four separate trans-mountain diversion projects in Colorado. Two of the largest water-diversion projects feature long tunnels that cross under the Continental Divide. The Colorado-Big Thompson Project moves water from Grand Lake to Estes Park through the 13-mile (21-km) long Alva B. Adams Tunnel under Rocky Mountain National Park, and the Dillon Reservoir project moves water 23 mi (37 km) through the Harold D. Roberts Tunnel, from Dillon to Grant.

Engineering geologists play a prominent role in surface-water storage and diversion projects. This role includes, but is not limited to, locating favorable sites for dams and tunnels, assessing geologic conditions prior to and during construction, and providing ongoing inspections of the tunnels and other component systems. Two chapters and several papers in this volume are dedicated to tunneling (see “Tunnels/ Underground Construction”) and dams (see “Dams/Water Resources”).

Hot Springs

Colorado has 93 known geothermal areas including natural springs, augmented natural springs, and wells, as well as several hundred smaller springs and seeps (George, 2000). Colorado’s geothermal resources are classified as “low temperature” – less than the boiling point of water – and are used primarily for recreational bathing, aquaculture, minor space heating, and heating green houses. Probably the most famous hot springs are those around Glenwood Springs. The springs issue from Mississippian limestone and are the saltiest in the state. Scientists believe that the high salinity of the springs around Glenwood Springs is related to dissolution of salt in the nearby Pennsylvanian evaporites (see the following section, “Geology and Water Quality”).

The hottest springs in the state are around Mount Princeton, with temperatures of up to 181°F (83°C) (Cappa and Hemborg, 1995). Other commercialized hot springs establishments in Colorado, including those at Idaho Springs, Pagosa Springs, Steamboat Springs, Hot Sulfur Springs, Poncha Springs, and Ouray, are well known for their popular spas. Many of these geothermal areas are associated with the Rio Grande Rift in central Colorado. Barrett and Pearl (1978) provide geologic, hydrologic, and geothermometer-analysis descriptions for many of the state’s major hot springs and geothermal resources.

Wetlands

Wetlands are areas that contain seasonally or perennially saturated soils and specialized, water-loving plants. Once reviled as unusable and insect infested swamplands, wetlands are now valued as being ideal for wildlife habitat, ground-water storage, flood attenuation, stream-bank stabilization, heavy metal and sediment retention, and recreation. Less than two percent of Colorado’s land is made up of wetlands. The state has lost about half of its wetlands in the 150 years since its pioneer days (Jones and Cooper, 1993).

In Colorado, wetlands take many forms including hillside seeps and slope wetlands, peatlands (fens), wet meadows, snowmelt depressions, marshes, ground-water flats in closed basins, and riparian wetlands along floodplains. Colorado’s largest wetland area is found in the San Luis Valley at the San Luis Lakes near the Great Sand Dunes. This closed-basin, groundwater flat contains many playas and marshes and is an important wildlife management area.

In recent years, people have begun to understand that there is an intrinsic link between geology and wetlands (see Brinson, 1993; Noe et al., 1998). Colorado’s wetlands exist because of the topography created by past and present geologic events and processes (Figure 10). They depend on off-site sources of water and water pathways that are controlled by topography and subsurface geology, and the hydrodynamics of the watershed. In sum, the health and viability of the state’s

Figure 10. The Colorado River has its headwaters in a wetland that occupies formerly glaciated terrain at LaPoudre Pass, in Rocky Mountain National Park.

wetlands depends not only on managing the wetlands themselves, but also on managing and maintaining the water sources and pathways within the surrounding watershed areas.

GEOLOGY AND WATER QUALITY

Overall, the quality of Colorado's surface and ground waters is very good. Both natural and human-caused processes, however, affect water quality. Many of Colorado's streams are subject to natural degradation from high concentrations of metals, salts, and other elements that occur naturally as a result of the local geology.

Dissolved Metals and Acid Rock Drainage – Natural acid rock drainage has been active in Colorado for at least thousands, possibly millions, of years. Acid rock drainage occurs when water and oxygen interact with metal-sulfide minerals such as pyrite, producing sulfuric acid, which dissolves metals and carries them into ground water and streams. The North Fork of the South Platte River in Park County, South Fork of Lake Creek in Lake County, and the upper Alamosa River in Conejos County are prime examples of this type of natural degradation (Figure 11). In many instances, metals in Colorado's headwater streams are derived from both natural and mine-induced sources.

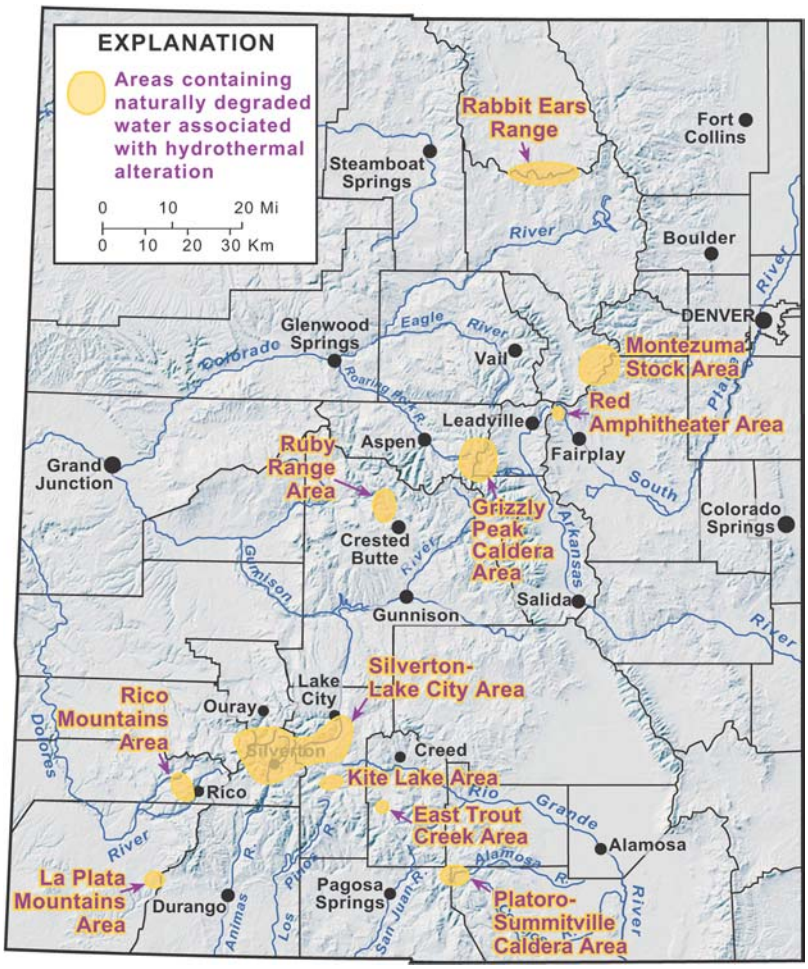

Figure 11. Areas of natural acid rock drainage in Colorado. From Matthews and others (in prep.); modified from Neubert (2000).

Evaporites and Salinity – High salinity concentrations are a concern for water quality in the Colorado River basin. The causes are varied, but geology is an important factor. Thick deposits of ancient sea salt, in the Pennsylvanian Eagle Valley Evaporite, underlie parts of the river basin in the Eagle-Glenwood Springs area. Ground water and surface water continually dissolve this salt body. Recent geologic mapping by the Colorado Geological Survey indicates that this dissolution process has been going on for several million years and, over this span of time, as much as 550 mi^3 (2,300 km^3) of salt deposits may have been carried away by streams (Kirkham and Scott, 2002). Natural hot springs, such as those near Glenwood Springs and Dotsero, carry especially large amounts of dissolved evaporite material and aggravate the salinity problems of the Colorado River. Cretaceous shale formations in western Colorado, particularly the Mancos Shale, also contribute to the salinity problem.

Shales and Selenium – Selenium, an essential nutrient required for human health, can be toxic in high concentrations. Elevated levels of selenium occur in several Colorado streams. The selenium comes from Upper Cretaceous marine shales, particularly the Mancos and Pierre Shales. When residual soils over these formations are irrigated, the water dissolves selenium and transports it to nearby streams. Areas of concern in Colorado include the Gunnison River basin, Grand Valley, Pine River basin, and Middle Arkansas River basin. Certain reaches of these streams have selenium concentrations above the national maximum contaminant level of five micrograms per liter.

Radioactive Elements – Naturally occurring radioactive elements can occur in ground water in many areas of the state and are a health risk in high concentrations. Radon, radium, and uranium are found in small amounts in most rocks. If rocks that have greater than normal concentrations of these elements comprise an aquifer, the local ground water may contain unacceptably high levels of radioactivity. For example, individual water wells in certain areas underlain by the Cretaceous-Tertiary Dawson Formation in Douglas County, and by Pennsylvanian to Cretaceous formations in Park County are known to have high concentrations of radioactive elements.

Depending on the radiochemical makeup of the water, different types of filtering or treatments may be necessary, at varying costs. Filtering treatments may neccesitate difficult waste-disposal considerations. The blending of more- and less-radioactive waters from different wells has been used with some success near Castle Rock and in other areas. High radon concentrations may be treated by bubbling or other methods (James Jehn, 2003, pers. comm.). Water wells should be tested for gross Alpha, gross Beta, and uranium concentrations, all of which have state drinking-water-quality standards. There are currently no standards for radon concentrations.

Human Interactions – Human interactions with the local geology are another important cause of water degradation. Mine waste and tailings piles that expose sulfide minerals are an example that dates from Colorado’s history; the irrigation of selenium-bearing shales for historic and modern agriculture use is another. In the mountains, crystalline Precambrian rocks may host fractured-rock aquifers that have very limited storage. As households multiply in these areas, creating increased demands from many wells and increasing the number of on-site waste disposal systems, such local aquifers may be quickly depleted or polluted. This is an important issue in the mountain suburbs to the west of Denver and in many other areas in Colorado.

Development in floodplains and stream-channel modification also impact water quality by increasing erosion and siltation, and by introducing chemical pollutants. Modifications that result in the loss of native riparian vegetation or wetlands will not only reduce habitat, but will impair the natural treatment capacity of the vegetative system. Significant impacts related to land use may include urbanization, agriculture, road construction and maintenance, mining, and timber harvesting. Planning for such activities is essential for maintaining water quality.

Geology plays an important role in water quality. Some water quality issues in Colorado have their foundation in the natural interaction between water and rock. This interaction can produce poor water quality independent of other influences, but existing problems are often exacerbated when people become part of the equation. The geology and water quality of an area should be assessed before any development takes place – this will alert planners to potential problems.

GEOLOGIC HAZARDS

Colorado's population has increased from 3.3 to 4.3 million persons from 1990 to 2000. Ten metropolitan counties along the Front Range accounted for over 80 percent of that increase. Although the Front Range urban corridor houses the majority of Colorado's population, the central mountains and the western slope were the fastest growing sub-areas of the state in the 1990s. This accelerated growth has focused public attention on the natural resources required to sustain the state's wildlife, communities, and business.

Less recognized in the public eye, and poorly understood by most Coloradoans, geologic hazards and their associated planning and mitigation issues are becoming increasingly critical as the state's population continues to grow. Nearly all of Colorado's populated areas are subject to geologic constraints and hazards of one form or another. Geologic hazards, by definition, are naturally occurring or human-influenced geologic processes that constitute threats to health and safety and property. Geologic constraints are geology-related conditions that may pose less of a threat, but may still be capable of causing damage. Both geologic hazards and constraints should be addressed as part of the planning and construction of any development project.

Engineering geologists perform a major role in the recognition and mitigation of geologic hazards. The primary duty of the Engineering Geologist is the site investigation, in which geologic hazards and constraints are identified, characterized, and reported. Equally important is the engineering geologist's interaction with project planners and developers, who must design and lay out the project so as to avoid or minimize the hazard areas, as well as the geotechnical, civil, and structural engineers who must produce the mitigative designs and parameters. If an engineering design is the answer to a problem, then engineering geology is the means of defining the correct question that must be answered, or the correct geologic problem that must be solved. Finally, engineering geologists are responsible for observing and assessing actual site conditions during construction. Such assessments are needed in order to ascertain whether the planned mitigative designs remain appropriate or whether design changes are necessary.

The cost of addressing geologic hazards in the preliminary phases of a project, using proper engineering geologic site investigations and planning, is less that the cost of making changes later in the project when unplanned-for actual conditions dictate that the plans must be changed during construction. The cost increases markedly if ground movements are not properly planned for, and subsequent movements, deformations, and failures occur and remedial actions become necessary. The benefits of a proper and sufficiently detailed, engineering geology investigation, and the associated lowering of risks at a site, are enjoyed by owners, contractors, and (for public projects) users and the tax-paying public at large.

Colorado is subject to a wide variety of potentially destructive geologic hazards. An abbreviated introduction to these hazards is included in the following section. Many of the technical papers in this volume are focused on a particular type of geologic hazard, or are case studies that focus on a suite of geologic hazards and constraints at a particular location.

Expansive Soil and Bedrock

Expansive soil and bedrock constitute Colorado's most costly geologic hazard in terms of damage to buildings, roads, and infrastructure. There are widespread surface exposures of these materials across the state. This is due, in part, to the tendency of non-resistant, clay-bearing materials to weather into relatively flat, low topography. Thus, most of Colorado's populated areas are "conveniently" located in areas of expansive soil and bedrock on the eastern plains (Figure 12) and along the valley bottoms in the central mountain and western plateau provinces. The major expansive bedrock units include the Cretaceous Pierre Shale, Mancos Shale, and Laramie Formation, the Cretaceous to Paleocene Denver and Dawson Formations, and the Jurassic Morrison Formation.

Figure 12. These suburban areas near Denver are encroaching into areas where expansive soil and heaving bedrock are potentially destructive geologic hazards.

Expansive soil problems have been the subject of intensive study in the Denver area since the 1950s. Many innovative approaches to mitigating these soils have been developed by area engineers. One of the more persistent and destructive problems pertains to the case of expansive, steeply dipping bedrock in the southwestern suburbs of Denver. There, significant amounts of differential ground heaving has caused extreme damage to facilities, confounding traditional engineering approaches for over 20 years. It was not until the mid-1990s that publicly available studies described the physical framework of the subsurface geology and the differential-movement mechanisms. Subsequently, at the request of some of the local, county and municipal governments, an integrated engineering-and-geologic approach to site exploration and facility design was formulated and adopted (see Noe, 1997). This approach has met with considerable success to date.

In their natural state, Colorado's expansive soil and bedrock are typically dry in near-surface exposures as a function of the state's dry climate. Development in such areas increases the amount of moisture in the ground as a consequence of lawn irrigation, cutoff of evaporation by impervious surfaces, and poorly executed surface drainage. These areas undergo swelling after development. Only recently, during a period of severe drought, has another expansive soil mechanism – shrinking – become a factor in already developed areas. This has become apparent as several of Colorado's municipalities have placed restrictions on water use and lawn irrigation.

There are a number of technical papers about expansive soil and bedrock hazards in Colorado in the "Expansive and Collapsible Soil and Bedrock" chapter of this volume.

Collapsible Soil

Vast areas of Colorado are underlain by silt-bearing soils that are typically dry, but which are prone to collapse and settlement when wetted. These soils have high amounts of void space due to disturbance, dissolution, or rapid deposition in a number of depositional environments. Examples include residual soils overlying the Mancos Shale in western Colorado, and widespread landslide, debris flow, eolian sand, and loess deposits throughout the state.

Some of these deposits are gypsiferous and prone to in-place dissolution and void formation. Piping of silty soils also forms large voids and cavities. Certain collapsible soils contain clay particles that may be prone to swelling as well as void collapse. Many western Colorado towns have experienced soil-collapse problems including Montrose, Delta, Grand Junction, and Meeker. Several Front Range communities in the Denver, Colorado Springs, and Pueblo areas have experienced these problems as well.

Collapsible soil hazards in Colorado are the subject of a technical paper in the "Expansive and Collapsible Soil and Bedrock" chapter of this volume.

Evaporite and Limestone Karst

The Pennsylvanian Eagle Valley Evaporite in west-central Colorado is a partially extruded deposit that is exposed in several valley-centered salt anticlines. Although the near-surface salts have been dissolved, the gypsum deposits remain and are exposed on or near the valley floors. These deposits are prone to forming karst topography when water sources are introduced, through processes of local dissolution and sinkhole formation, and can be subject to incremental or sudden collapse as a result. This is an increasingly important land-use issue for rapidly developing towns including Glenwood Springs and Carbondale in Garfield County and Eagle and Gypsum in Eagle County, and for nearby, unincorporated suburban developments.

Although Mississippian-age karsting of the Leadville Limestone has been well documented, not much is known about modern karsting within this unit. Modern dissolution is indeed occurring, as evidenced by the many caves found within the limestone (see "Caves," in the earlier section on "Quaternary Geology," in Part I). It may be that the near-surface exposures of the Leadville Limestone are sparsely settled, resulting in a low exposure to karst-related hazards.

Evaporite karst hazards in Colorado are the focus of two technical papers in the "Expansive and Collapsible Soil and Bedrock" chapter of this volume.

Coal Mine Subsidence

Many of Colorado's older, abandoned coal fields contain shallow workings that are prone to collapse, causing subsidence of the ground surface. Many of the original mining towns avoided the areas over underground workings. Today, several of these towns along the Front Range Piedmont are growing significantly as bedroom communities, and there is development pressure to build over undermined areas. Coal mine subsidence is an important land use issue in Colorado Springs and the Jefferson County and Boulder-Weld coal fields. Coal is currently mined in northwestern Colorado, where the topography is much more rugged. There is a lower overall subsidence hazard because the mines are seldom located near populated areas, and because many mines are surface mines.

The modern, underground mines use continuous longwall methods instead of the older room-and-pillar mining, resulting in more uniform and immediate subsidence of the ground surface over deep mines. Stepwise ground subsidence at the shallow Foidel Creek (Twentymile) coal mine, near Hayden, has resulted in spectacular rockfall as a massive cliff of Cretaceous Mesaverde sandstone has undergone incremental subsidence and failure (Figure 13). This human-caused process is located far from any town site, and the resulting impacts to the property and the nearby county highway were considered and designed for as part of the mine's extraction plan.

A more-extensive technical paper on the topic of coal mine subsidence is included in the "Mine Development and Remediation" chapter of this volume.

In addition to subsidence hazards, burning underground coal seams are found in numerous locations across the state. These fires have been initiated by spontaneous combustion of coal dust, lightning, and other means. At least 29 coal fires exist in areas of abandoned, older coal mines (Rushworth et al., 1989), while others may exist in areas of natural outcroppings. Some have been burning for as long as a century. A long-lived coal fire in the Marshall area, near Boulder, and associated coal mine subsidence have placed constraints on nearby development. In 2002, a long-lived coal fire near Glenwood Springs that had been burning since at least 1910 attracted national attention by being identified as the cause of a 27,000-acre forest fire. The Colorado Division of Minerals and Geology has a program to monitor and, where possible, extinguish coal mine fires in Colorado. This is often a costly and difficult effort.

Landslides, Rockfall, and Debris Flows

Landslides, in the most general sense, are features that result from the downslope movement of rock, soil, and other debris. Gravity is the main driving force for all landslide types. Water is a driving force as well; it adds weight to the soil or rock mass and, under saturated conditions, can

Figure 13. This cliff of Mesaverde sandstone is undergoing subsidence from underground coal mining, resulting in spectacular rockfalls. Photo by Jonathan White, Colorado Geological Survey.

induce buoyant forces that lessen internal shear strength and resistance. Rockfall is a specific type of landsliding that involves gravitational breakaway and rolling of individual rocks or groups of rocks. Debris flows are another specific type landsliding that involves water-driven transport of rock and sediment and other debris as semi-solid debris plugs or as a hyper-concentrated flow. Some of the longer rockfall and debris flow paths have a characteristic morphology that consists of an initiation zone, a steep, central acceleration zone or chute, and a fan-like runout zone at the base (Figure 14).

These three related geologic hazards are found in areas of moderate to steep topography. This includes many areas of Colorado's mountains and plateaus, and isolated areas on the plains. Large landslides are found in virtually every rock formation, especially the Precambrian igneous and metamorphic rocks, late Paleozoic red beds, Cretaceous shales, and in Tertiary sedimentary and volcanic rocks. The transported or residual soils that overlie these formations may host landslides as well. Colorado's landslides include translational landslides, in which blocks of rock slide upon bedding or fracture or joint surfaces, and rotational landslides, which form in more homogeneous or soil-like materials. Landslides have impacted many of the state's communities, particularly Colorado Springs, Lakewood, and Grand Junction, in recent years.

Debris flows and rockfall affect several Colorado communities. Towns such as Georgetown, Glenwood Springs, Aspen, Marble, and Ouray are built on large, coalescing debris-flow fans and have experienced damaging flow events during their histories. Recent rockfalls have damaged structures in Vail and Telluride. Colorado's highway system is subject to serious debris flow and rockfall hazards. Recent geologic research into alluvial-fan stratigraphy along the I-70 Corridor

Figure 14. Typical debris flow path in west-central Colorado, near Eagle, showing a well developed initiation zone, a central chute, and runout fan. Photo by David Noe and Jonathan White, Colorado Geological Survey.

has revealed a significant connection between wildfire events and subsequent debris-flow and alluvial-fan flooding activity (Coe et al., in press). This relationship has been illustrated dramatically in several areas of the state where numerous significant post-wildfire floods and debris flows have occurred during the 1990s and early 2000s. Recent events of note include those at Buffalo Creek to the west of Denver (Elliott and Parker, 2001), Storm King Mountain near Glenwood Springs (Cannon et al., 2001), and along the Animas River valley near Durango (Cannon et al., 2003, in this volume) (Figure 15).

The state has list of critical landslide, rockfall, and debris flow areas that is maintained and updated by the Colorado Geological Survey for the Colorado Office of Emergency Management. Rogers (2003) discusses the results of the latest update to the statewide critical-landslide list and map in a paper in this volume. The general topic of landslides and related hazards in Colorado, and their mitigation, is covered in several technical papers in the "Transportation" and "Landslides, Rockfalls, and Debris Flows" chapters of this volume.

Not all landsliding has led to undesirable results. Much of Colorado's famous, rolling ski terrain consists of landslide complexes that have partially filled older, formerly steep-sided, glacial valleys. Excellent examples of this are evident at Keystone, Breckenridge, and Vail ski areas. The resulting lower-slope topography is excellent terrain for beginner to intermediate skiing. Landslide complexes are often colonized by beautiful and much-photographed aspen groves.

Figure 15. House inundated by debris flows on a fan along the side of the Animas River valley, near Durango, which was reactivated following the 2002 Missionary Ridge wildfire. Photo courtesy of La Plata County Office of Emergency Management.

Thus, indirectly, landslides can be a boon to the state's tourism and economy when prudently assessed and utilized.

Stream Flooding

The largest stream flooding in Colorado's recent geologic history occurred during the Pleistocene and Holocene ice ages, as a consequence of episodic melting of the great alpine glaciers. Although smaller by comparison, stream flooding continues to have an important impact on Colorado's floodplains today. The two main, natural sources of flooding in the state are from snowmelt and precipitation events. Given Colorado's semi-arid climate, which features inconstant snowfall amounts from year-to-year, and even more inconstant precipitation from year-to-year and storm-to-storm, its flood history is erratic and flood forecasting is difficult.

The snowmelt-caused flooding usually involves a seasonal, long-period rising and falling of stream water levels. This affects many of Colorado's smaller, higher-altitude streams, often on a yearly basis. In contrast, the precipitation-caused flooding is often a result of single rainstorms or short, multi-day storm events. All parts of Colorado are prone to orographic (topography-driven) thunderstorms, which may release prodigious amounts of rainfall over a small area over a short period of time. Rainfall amounts of 12 in (30.5 cm) or more are not uncommon for some of these large storm events. A particularly destructive combination is a large rainfall event over a drainage basin that has an already-high runoff from upstream snowmelt.

Although most areas have experienced stream flooding, there are two areas of the state where the flooding hazard is especially pronounced. One is the Palmer Divide, an upland area on the plains between Denver and Colorado Springs. Historically, this area is subject to intense thunderstorm activity during the summer. It has been the source area for some of Colorado's largest flood events, including the 1965 Plum Creek/South Platte River, and Bijou Creek floods. The other area is the Front Range, where numerous streams have cut narrow canyons in the Precambrian granites and gneisses. Major highways follow these canyons. In addition to use by summer tourists, an increasing number of commuters who have moved into the mountains now travel many of these canyons daily. Mountain-torrent flash floods from thunderstorms along these stream corridors have created disasters along these canyons and in towns on the plains below the canyon mouths, such as Loveland, Boulder, and Morrison.

Colorado's worst, modern-day flooding disaster was the July 1976 Big Thompson Canyon flood. An evening thunderstorm dumped 12 inches of rain over a small area below Estes Park, resulting in a flash flood that left 139 people dead and caused $40 million in property damage to roads, residences, and motels. The peak flow was 39,000 cfs (1,104 m^3 per second), with a water level of 14 ft (4.3 m) above normal in the Narrows, near the mouth of the canyon. As a result of post-flood investigations, the U.S. and Colorado Geological Surveys (Soule et al., 1976; Shroba et al., 1979) found that much of the destruction and geomorphic impact to the area occurred outside of the modern floodplain. These impacts were caused by local processes that involved sheetflow, rill erosion, deep scouring in side canyons, debris flows, landslides, and sediment deposition.

In Colorado, the Colorado Water Conservation Board is responsible for flooding issues including floodplain delineation, regulation, emergency response, and planning. Geologic input is useful for many of these issues, and may include such items as geomorphic assessments of past and recent flood flows, geology-related aspects of watershed erosion and stream-sediment yield, and delineation and modeling of debris-flow and alluvial fans.

Avalanches

Colorado's high mountain areas are subject to avalanches – gravity-driven flows of snow, ice, and other debris – during the winter and spring months. Avalanches occur as singular incidents, and their initiation is controlled by a variety of factors, including topography, aspect, snow conditions, and weather conditions. They may be as small as a few hundred square feet or as large as an entire mountainside. Many avalanche areas have events that recur year after year or after significant snowfalls. Some of the longer avalanche runs, especially those that begin above timberline and run into steep-sided, glacial valleys have a morphology similar to that of debris flow areas, consisting of a bowl-shaped initiation zone, a steep, central acceleration zone or chute that lacks woody vegetation, and a fan-like runout zone at the base (Figure 16). Debris flows and avalanches may share the same paths, thus constituting a year-round hazard.

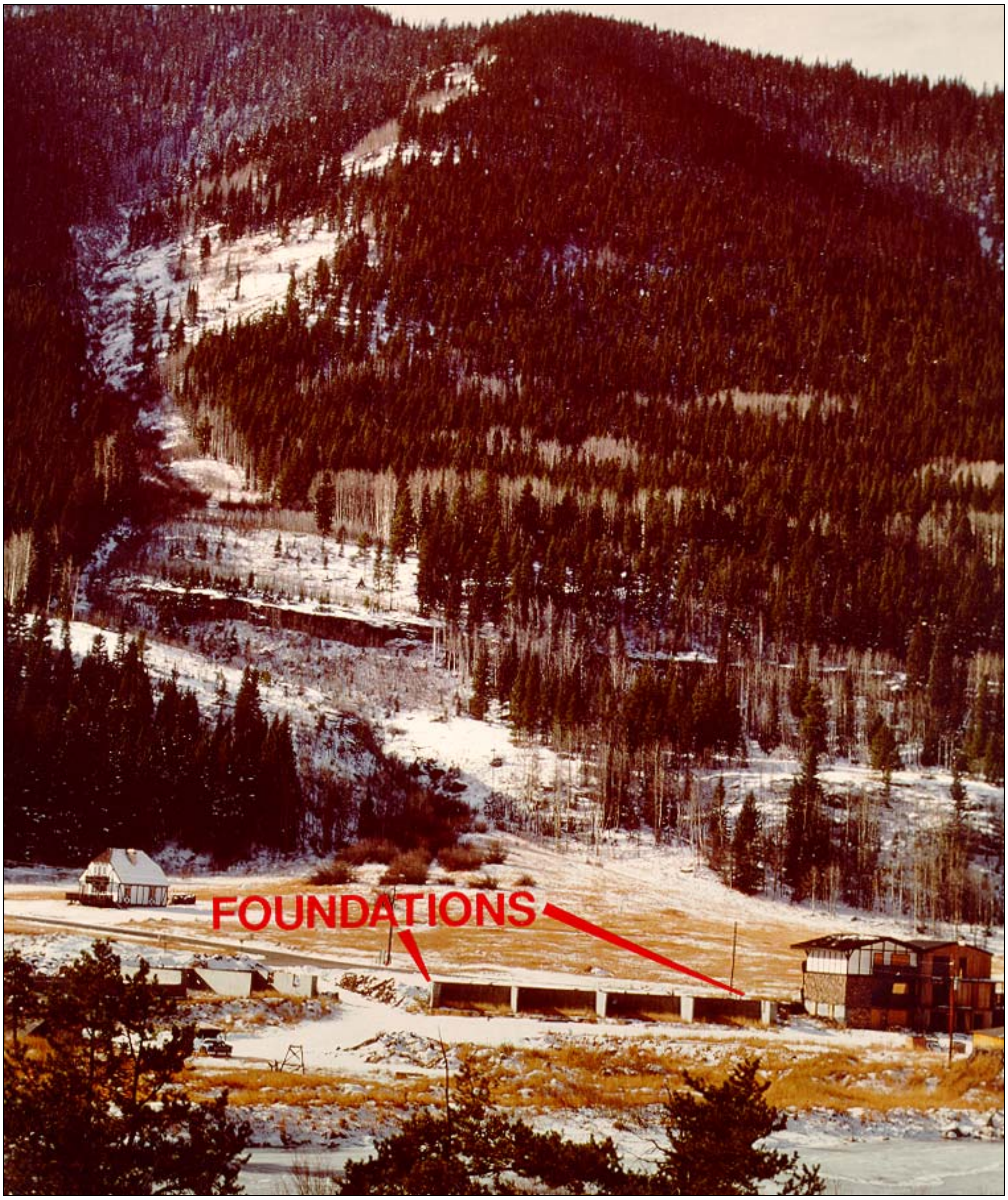

Figure 16. An avalanche path in Vail, Colorado. A forested initiation zone and a well-developed central chute and runout zone are visible. Houses within the runout-zone fringe have been built with reinforcement for design-avalanche loads. Photo from Colorado Geological Survey.

The historical tracking of avalanche events and the recognition of avalanche-prone terrain makes it possible to model and predict their behavior and to map the extent of the hazard. Because of the extreme forces exerted by avalanches and the extreme destruction they can cause, avoidance is a primary consideration for avalanche areas. Equally important is defining the extent of less hazard-prone zones at the fringes of the high-hazard runout area (Mears, 1992). In these zones, it may be possible to locate facilities based on careful mitigation strategies.

In Colorado, certain townsites such as Silver Plume, Vail, Marble, and Mount Crested Butte encroach upon avalanche-prone areas. Some of these towns have instituted avoidance strategies out of necessity, while others have developed or inhabited areas that remain at risk. Some of the older mining towns, like Silver Plume, have been the sites of repeated, episodic avalanche incidents accompanied by loss of structures and deaths (Martinelli and Leaf, 1999).

In addition to inhabited structures, avalanches pose a threat to travelers on the state's mountain highways and to backcountry travelers. Approximately 300 avalanches block Colorado's highways each winter. Colorado has the highest annual death rate (about 6 people per year, on average) from avalanches in the entire United States. The Colorado Avalanche Information Center (CAIC) is charged with the responsibility of forecasting statewide avalanche conditions, issuing warnings and alerts, giving avalanche-safety training courses, and working with the Colorado Department of Transportation and ski areas to reduce human exposure to avalanches. CAIC operates a Web site at http://geosurvey.state.co.us/avalanche/.

Earthquakes and Seismicity

More than 500 earthquakes of various magnitudes have been recorded in Colorado during historical times (Figure 17). The largest earthquake was in 1882, located near Estes Park, and is estimated to have been greater than magnitude 6.6. This earthquake knocked out power in Denver and was felt as far away as Salt Lake City.

Colorado has had several instances of notable, human-induced earthquakes. During the 1960s, the Denver area experienced over 200 earthquakes, three of which reached magnitudes of greater than 5.0 in 1967. Subsequent investigations of these earthquakes found that they were triggered by the 2-mile deep injection of waste fluids by the U.S. Army at the Rocky Mountain Arsenal. Once the waste-disposal operation was curtailed, the number of earthquakes dropped off dramatically (Evans, 1966, Hollister and Weimer, 1968).

In northwestern Colorado, the U.S. Geological Survey studied earthquakes at the giant Rangely oil field that were related to fluid injections. They found that the earthquakes could be turned on and off repeatedly by varying the injection pressures (Gibbs et al., 1972). In 2000, the U.S. Bureau of Reclamation began injecting brines deep into the ground in the Paradox Valley of western Colorado, in an effort to reduce the salinity of water entering the Colorado River. This operation generated over 3,500 small earthquakes, and was cut back when one earthquake exceeded magnitude 4.0.

There are at least 272 faults and 27 folds in Colorado that show evidence of movement from late Cenozoic time to the present (Widmann et al., 2000). Of these, at least 92 faults and 6 folds show evidence of movement during the Quaternary Period (Widmann et al., 1998) (Figure 18). The geologic characteristics of several of these faults indicate that they are capable of generating large earthquakes in the future. Earthquake studies in Colorado are still in their infancy, compared to many other states, and much study remains to be done regarding the risk of a major earthquake affecting populous areas of the state. Several papers on the topics of earthquakes and seismicity in Colorado are found in the "Transportation" and "Faulting/Earthquake Hazards" chapters of this volume.

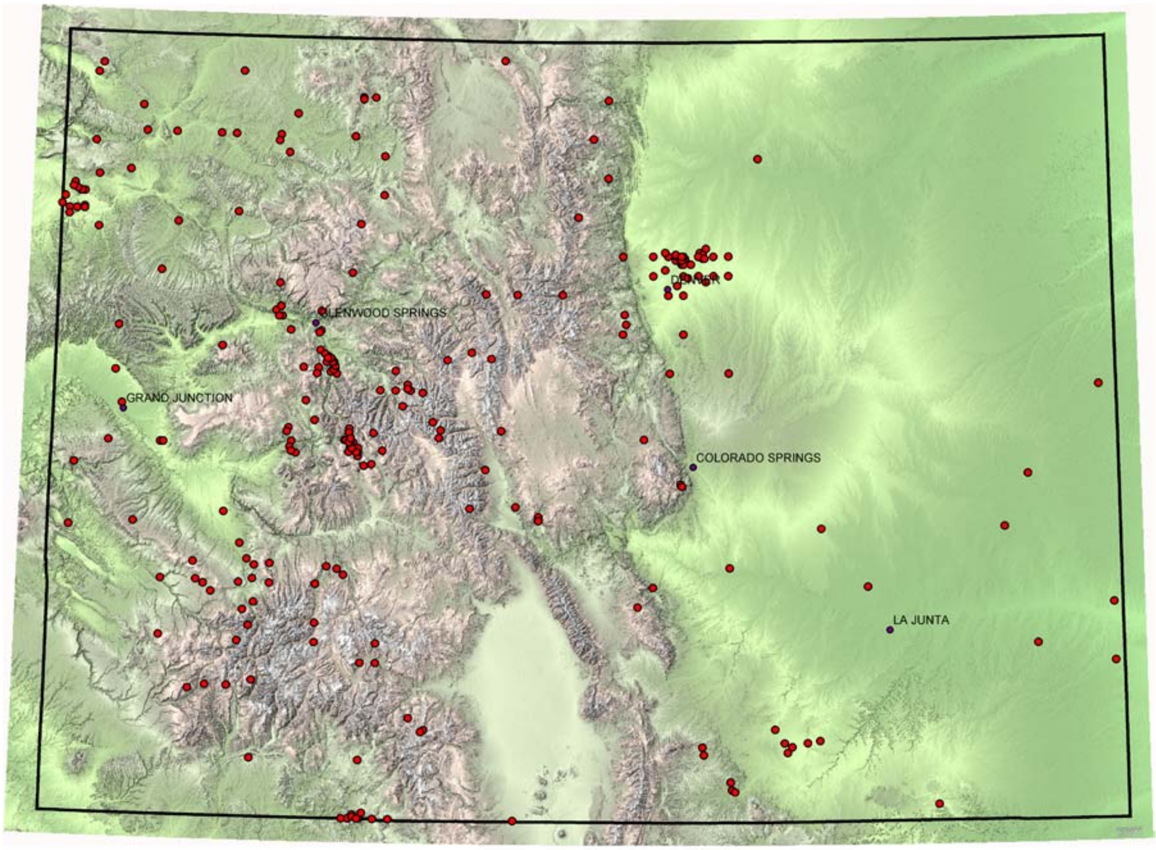

Figure 17. Historical earthquakes in Colorado, including the 1882 earthquake. From Matthews and others (in prep.); modified from Kirkham and Rogers (2000).

Radon

Radon is a colorless, odorless gas that is a bi-product of the radioactive breakdown of uranium minerals. Many of Colorado's geologic formations, particularly the granites and shales, and residual and transported soil deposits derived from these formations, contain elevated levels of radon. A study by the Colorado Geological Survey (1991) found that many areas of the state exceed the 4 pCi/l limit for indoor radon concentrations set by the U.S. Environmental Protection Agency. In practice, proper venting of "tight" structures may mitigate radon. Many builders in Colorado choose to not install vents, however, leaving it up to future owners to test for radon and, if necessary, install retrofit venting systems.

Environmental Hazards

Geologic weathering processes can release a variety of metals, salts, and other elements into the environment that degrade the soil and water and may be harmful to humans and other organisms. The effect of local geology on surface- and ground-water quality in Colorado is discussed in the previous section ("Water Resources and Water Quality"), and is the subject of several papers in the "Ground Water/Environmental" chapter of this volume.

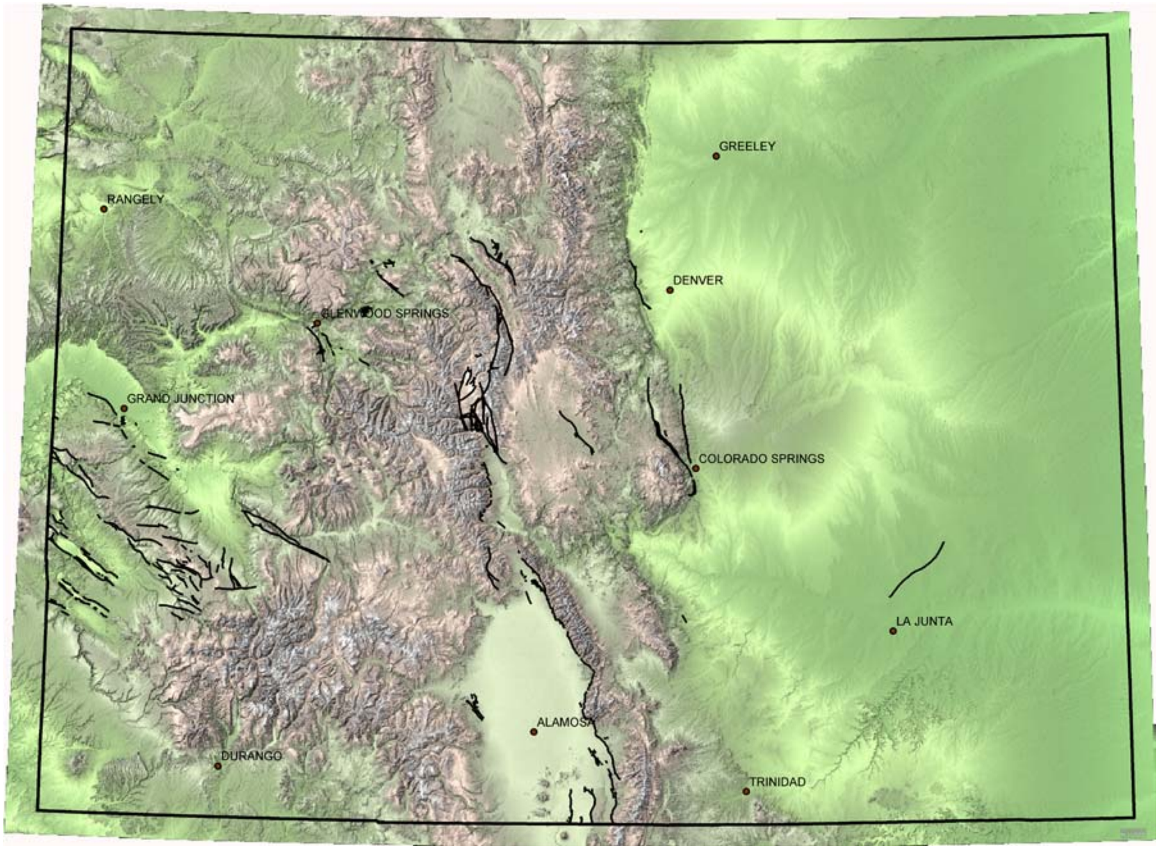

Figure 18. Distribution of Quaternary faults in Colorado. From Matthews and others (in prep.); modified from Widmann et al. (1998).

Land Use Laws

Colorado has enacted several laws that require the identification, mitigation, and disclosure of geologic hazards. Other state laws require the identification of and planning for mineral resource areas. In Colorado, the Colorado Geological Survey is responsible for providing geologic-suitability reviews of technical documents that are submitted to counties, municipalities, school districts, and special districts for development projects.

A description of these land-use laws and other local government ordinances, regulations, and codes, and a discussion of CGS review activities, is contained in a paper by Berry and Noe (2003), in this volume. Two other papers on the incorporation of geologic hazard assessments as part of land use planning in two Colorado counties are included in the "Land Use Planning" chapter of this volume.

The scope of any particular engineering geologic assessment for a land use project will vary, depending on the stage of planning, the type and layout of the project, and the nature of the local geologic conditions. Colorado's statutes do not address topical requirements for the scope of investigations or the content of geologic reports, and because there is no registration program for Profession Geologists in Colorado at this time, there are no applicable oversight documents.

Useful information and general guidelines about what to include in engineering-geologic site assessments and reports are available in Appendix E of CGS Special Publication 12 (Shelton and Prouty, 1979), and in web sites from states where registration of Professional Geologists has been enacted. In addition, many counties and municipalities in Colorado have unique and specific reporting requirements for land-use investigations and reports.

CONCLUSIONS

1) Colorado contains a wide variety of minerals and mineral fuels. The state's resources include precious and base metals, oil and gas, oil shale, coal and coalbed methane, radioactive minerals, industrial and construction minerals, and gemstones and ornamental stones.

2) Colorado has finite surface water and ground-water resources. Ground-water resources currently supply approximately 18 percent of the state's water needs, and development of this resource is continuing at a fast pace. Colorado's principal aquifers include Quaternary-age alluvial aquifers, basin-fill sediments, sedimentary rock aquifers, and volcanic and crystalline rock aquifers.

3) Some water quality issues in Colorado have their foundation in the natural interaction between water and rock. This interaction can produce poor water quality independent of other influences, but existing problems are often exacerbated when people become part of the equation.

4) Geologic hazards and their associated planning and mitigation issues are becoming increasingly critical as Colorado's population continues to grow. Nearly all of the state's populated areas are subject to geologic hazards or constraints of one kind or another. These include expansive soil and rock, collapsible soil, evaporite karst, coal mine subsidence, coal-seam fires, landslides, rockfall, debris flows, stream floods, avalanches, earthquakes, radon gas, and environmental hazards.

5) Colorado's mineral and water resources, and its geologic hazards, are closely associated with the state's geography, surface and subsurface geology, geomorphology, and past and present geologic processes. An understanding of geology is crucial in order to wisely and efficiently develop Colorado's mineral and water resources, and to protect the public from geology-related threats to life, health, and safety.

ACKNOWLEDGEMENTS

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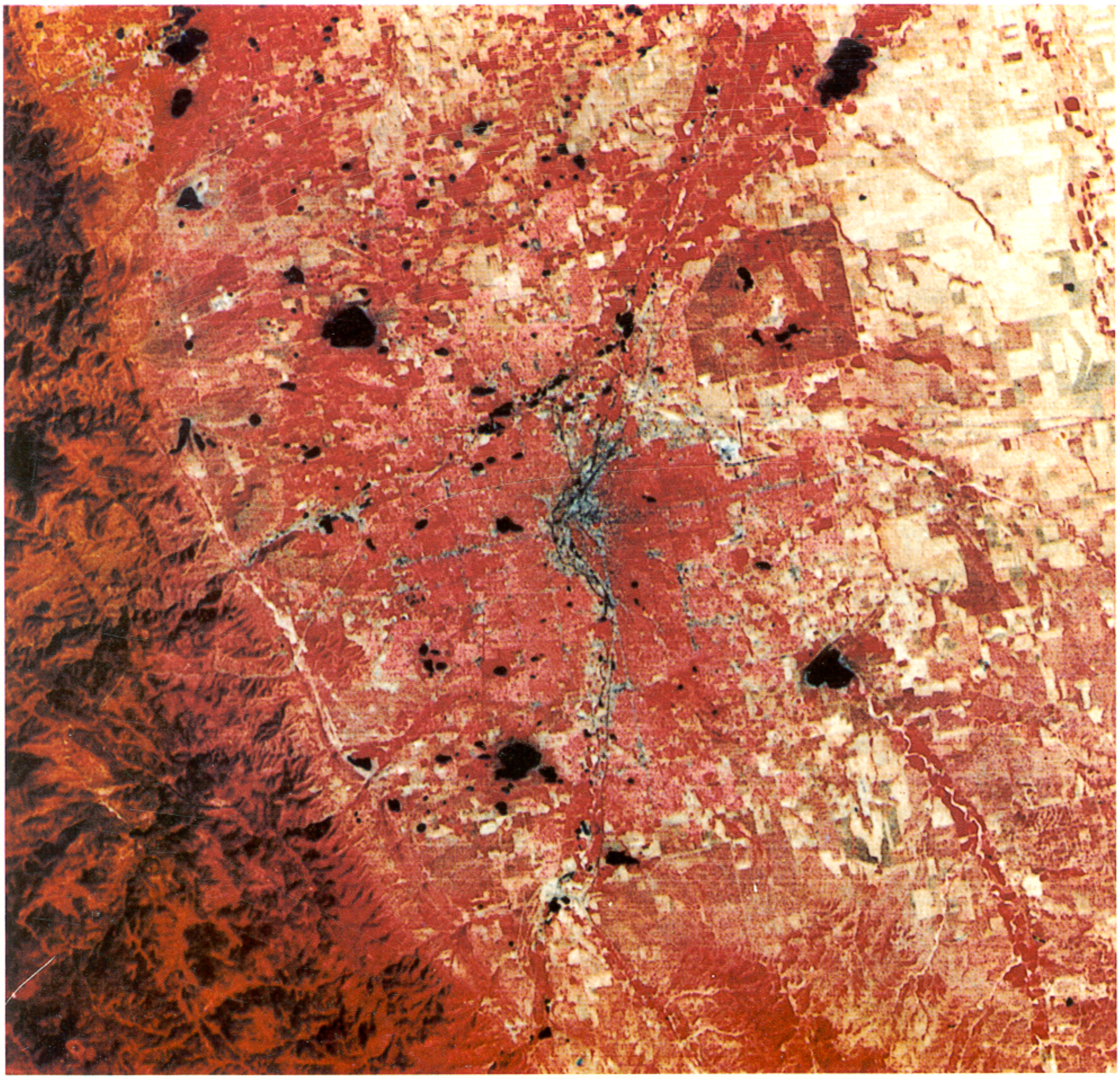
Geology of Denver, Colorado, United States of America



JOHN E. COSTA AND SALLY W. BILODEAU

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Cover Photo: Landsat false-color composite image (bands 4, 5 and 7) of the city of Denver and surrounding area, August 15, 1973. The image covers the area shown in Figure 1. Denver, the Queen City of the Plains, is in the middle of the image, and the boundary between the Great Plains and Southern Rocky Mountains is clearly seen along the western side of the image.

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Geology of Denver, Colorado, United States of America

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FOREWORD

The Executive Council and Board of Directors of the Association are pleased to provide the membership with a new series in the *Bulletin*, *Geology of The World's Cities*. Cities are the irrevocable focus of all that drives civilization forward; cities are the cauldrons that produce the pressures of cooperation and confrontation between peoples and nations; cities have been the birthplace of culture; cities have been the depletors of natural resources; cities have been the generators of immense quantities of wastes that now peril the environment. Cities, for all of their good and bad, are the fundamental aspect of human life on the planet.

The Association recognizes that each city was originally established for reasons of geologic influence. These same geologic influences are still present, both in the city's shape and structure and as constraints on what can and should be accomplished to prepare the cities for continued service in the coming centuries. In offering this series of papers, the Association hopes to discover elements of geologic influence and impact, so that the whole spectrum of practitioners can better control the renovation and rebirth of cities. By example of this series, peoples of various regions and nations will come to recognize that innovations of others have been applied to overcome some of the stresses on the people and resources of cities. To this end, we recognize the long-time influence of our distinguished Canadian colleague and native Briton, Dr. Robert F. Legget, who has labored in speech, text, and example for more than 45 years to bring this message to us all.

In this premier paper of the series, John E. Costa, an educator, and Sally W. Bilodeau, a practitioner, have presented the Geology of Denver, an American boomtown, grown large and commanding. Its presence, lodged at the eastern edge of the continent's greatest mountain range, marks the real transition from east to west in cosmopolitan America. Denver is the great North American city of our resource-conscious times. The great energies of Denver are people-generated and people-oriented. Denver runs on a 24-hour day, because it is the great sociologic magnet of the continent. Little of its humble frontier beginnings remain for detection by the casual visitor, but its origins are tied to its geologic setting: its development has been controlled by its geology; and its future will be guided by such influences.

Founded in 1858, on the site of placer gold discoveries, Denver has always served as a resource-oriented supply and operations center. Today the city serves a vast area of the central United States as a financial, engineering, scientific, governmental, educational and resource extraction center. The city that was born of resource extraction remains a key element in that activity today.

Denver's very existence, on the fringe of a great mountain range, displays the effect of the natural environment on the development of a city. Its near-region topography varies by nearly 8,000 ft (2,400 m); it lies on a sedimentary basin some 13,000 ft (3,960 m) thick; it consumes ground water and surface water at a phenomenal rate; it demands construction aggregates in alarming quantities; and it produces burdensome quantities of waste. Denver is affected by significant geologic constraints: both collapse-prone and swelling soils, hillslope instability, induced seismicity, flooding, and some areas of rising ground water. Denver is a city of the age and of the decade. The citizens and builders of Denver have learned to respect its geologic setting!

Papers in this series will be the result of cooperation between engineering geologists, geotechnical engineers, hydrogeologists, environmental engineers, seismologists, urban planners, and other allied technical specialists. Most of the papers will be released in the *Bulletin* along with other papers. Occasionally a group of cities in regional areas or nations will be printed in a single *Bulletin* issue. We welcome your continued interest in the series, both as concerned readers and as concerned authors.

Allen W. Hatheway, Series Editor,
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INTRODUCTION

Denver, known as the Mile High City, is the capital of the State of Colorado. The city is located in the west-central United States at latitude 39°44′N and longitude 104°59′W. The city center lies about 12 mi (19 km) east of the southern Rocky Mountains within the broad valley of the South Platte River and within the Colorado Piedmont section of the Great Plains geomorphic province (Figure 1). Denver itself has an area of 115 mi^2 (298 km^2) and a population of about 500,000. However, the Denver metropolitan area has a total population of 1.7 million (including Denver), and sprawls westward into the foothills of the Front Range and eastward onto the Great Plains. The term "Denver metropolitan area" refers to the core city of Denver and its surrounding suburbs, and is represented by the area in Figure 1.

History of Founding

The Denver area was originally occupied by American Indians at least 10,000 to 12,000 years ago. The land was claimed as French territory between 1682 and 1763, as Spanish territory between 1767 and 1800, and as French again between 1800 and 1803. Colorado became part of the Louisiana

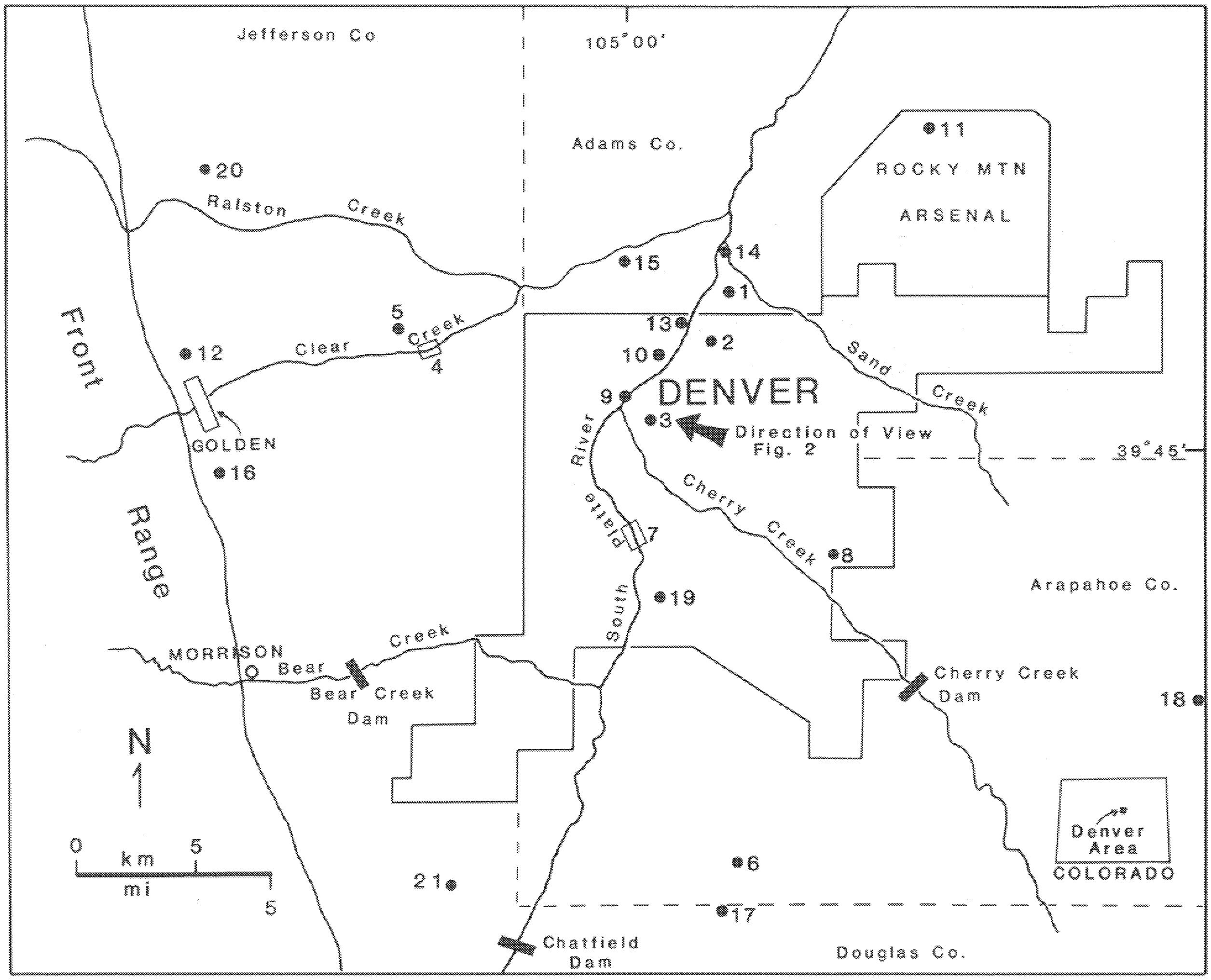


Figure 1. Location map of features in the Denver area discussed in the text. 1. Original site of Montana City; 2. Denver Coliseum; 3. Denver Hilton Hotel; 4. Reach of Clear Creek shown in Figure 13; 5. Ridge Home State School; 6. Isaac Newton Junior High School; 7. Reach of South Platte River shown in Figure 14; 8. Cedar Run Apartments; 9. Gaging station on South Platte River in Denver; 10. Regency Inn, location of strong motion seismograph; 11. Rocky Mountain Arsenal deep-disposal well; 12. Trench excavated across Golden Fault shown in Figure 19; 13. Denver Northside Sewage Treatment Plant; 14. Metropolitan Denver Sewage Disposal District No. 1 Plant; 15. Property Investment landfill; 16. Rooney Road landfill; 17. Arapahoe County landfill; 18. Lowry landfill; 19. Site of the National Radium Institute; 20. Leyden No. 3 Coal Mine used for natural gas storage; 21. Subdivision built over old underground Virginia coal mine. Arrow shows direction of view of photograph in Figure 2.

Territory purchased by Thomas Jefferson from Napoleon Bonaparte in 1803 for $15 million. The land also still belonged to the Arapahoe and Cheyenne Indian tribes, from whom it was eventually purchased in 1861 for $1.25 an acre (Mumey, 1942).

The original site of Denver was near the juncture of Cherry Creek and the South Platte River, where water was plentiful and willows and cottonwood trees offered shade, protection, and game (Figure 1). As early as 1820, it was a favorite camping place for some of the first white travelers through the region, including Colonel John C. Fremont, Kit Carson, and Major Stephen H. Long, and for Indians many years before that.

In the early 1850's, the discovery of placer gold along the South Platte River and its tributaries sparked interest in the area; and in 1858, a group of prospectors from Kansas laid out the first settlement, then called Montana City, on the east bank of the South Platte River about five mi (8 km) upstream from its confluence with Cherry Creek (Figure 1). Later in the year when the gold became de-

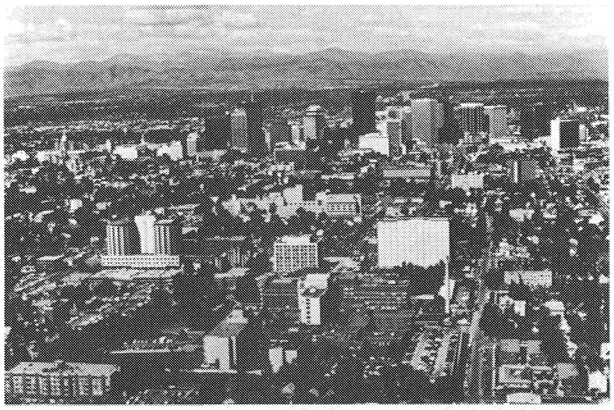


Figure 2. Oblique aerial view of Denver, 1980, looking west-northwest at downtown area. Southern Rocky Mountains in background. State Capitol is domed building at top left edge of photo. (Photo courtesy of Denver Planning Office.)

pleted, the people moved downstream to that confluence. There, in 1858, two towns were formed on opposite sides of Cherry Creek as more prospectors arrived. On the southwest bank was Auraria, named after the Georgia home of William Green Russell, an early prospector. On the northeast bank the town of St. Charles was founded; but only one log hut was constructed since most of the founding party returned to Kansas for the winter. In late 1858, another group of prospectors arrived from Kansas. They took over the townsite of St. Charles and renamed it Denver City after General James W. Denver, Governor of Kansas Territory. Denver was thus founded on a jumped claim. By the spring of 1859, the rival towns of Auraria and Denver had a total population of over 1,000. A year later in April 1860, they merged and adopted the name Denver City.

Placer gold deposits along the South Platte River and its tributaries were soon exhausted, but major discoveries were made in the mountains to the west of Denver. Early Tertiary intrusions 35 to 70 million years old, and rich in gold, silver, zinc, lead, copper, and molybdenum, were the major sources of mineral wealth. Denver grew primarily as a railhead and supply center serving the numerous mining towns in the mountains. The growth of Denver is documented by the rapid rise in population from 5,000 in 1870 to over 35,600 in 1880. Colorado became the 38th state in 1876; and five years later, in 1881, Denver became the state capital.

Denver faced two major crises in its early history. The first crisis occurred in 1866, when the Union Pacific Railroad announced its westward line from Council Bluffs, Iowa, would go through Cheyenne, Wyoming, and the Wyoming Basin, rather than to Denver and face the rugged southern Rocky Mountains with no easy westward outlet. Faced with the prospect of isolation, Denver built the Denver-Pacific Railroad in 1870 which joined the Union Pacific in Cheyenne.

The second crisis in its early history was caused by Denver's economy, which was one commodity—mining, especially silver mining. In response to recurring discoveries of silver in the mountains and the large federal market for silver, three large smelters and numerous companies specializing in mining equipment located in Denver. But in 1893, the Sherman Silver Purchasing Act was repealed as the United States shifted from a silver to a gold monetary standard. The price of silver crashed, and ten banks in Denver failed. William Jennings Bryan, a frequent presidential candidate, became well known for his speeches that the nation was being "crucified on a cross of gold."

Fortunately, a major gold discovery in Cripple Creek gave new life to Denver, and the city began to diversify its economic base. Agriculture on the Great Plains prospered, especially sugar beets, cattle, and sheep raising. The delightful healthy climate attracted more people; and by 1900, Denver had become a tourist center. Since 1945, skiing and other recreational activities have grown; and many federal and military agencies have moved to Denver. Large reserves of oil and gas were found northeast of Denver. Today, Denver is a major industrial, commercial, tourist, recreational, and governmental center in the middle of one of the fastest growing regions in the United States (Figure 2).

Climate

The Denver area is blessed with a semi-arid, temperate-continental climate (Trewartha, 1968) which is strongly influenced by the Rocky Mountains just west of the city. In winter, polar air moving southward is deflected east of the mountain front so that temperatures over the Great Plains can be much colder than temperatures in the mountains a short distance west. The eastward flow of Pacific air masses from the west is disrupted by the Front Range, causing heavy orographic snowfall in the mountains while Denver enjoys sunny skies and dry air. The mountains also block the northward flow of humid Gulf air masses from the southeast. This creates an easterly upslope circulation of air, a condition responsible for Denver's largest snowfalls in winter and heaviest rainfalls in spring and summer.

The mean annual precipitation is 13.8 in. (350 mm), but annual variations range from six inches to 23 in. (152–582 mm). Average annual snowfall is 55 to 59 in. (1,397–1,499 mm). Mean annual evaporation is 50 to 60 in. (1,270–1,524 mm) and the mean annual temperature is 52°F (11°C), ranging from a monthly mean of 70°F (21°C) in July to 28°F (−2°C) in January. Relative humidity averages 48 percent. Clear days (30 percent cloud cover or less) occur 30 to 60 percent of the time, and cloudy days (80 percent or more cloud cover) occur 16 to 36 percent of the time (Hansen et al., 1978).

Geologic Setting

Denver is located near the east front of the Southern Rocky Mountains in the Colorado Piedmont section of the Great Plains, the westward edge of the central stable area of North America. In this section, the Tertiary sedimentary cover that was deposited eastward onto the Great Plains from the erosion of the Rocky Mountains has been eroded by the South Platte and Arkansas River systems, exposing the underlying Cretaceous bedrock (Thornbury, 1965). The topography of the Colorado Piedmont is broadly rolling, with local scarps where resistant bedrock units outcrop. The land slopes from west to east at a gradient of about 10 ft/mi (0.0019 m/m) from 5,300 ft (1,615 m) in Denver to 4,000 ft (1,219 m) at the Kansas boundary.

To the west of Denver lies the Front Range of the Southern Rocky Mountains which extend for 185 mi (298 km) from southern Colorado into Wyoming. The Front Range is a complexly faulted anticlinal arch of primarily Precambrian crystalline rocks reaching elevations of over 14,000 ft (4,267 m) (Boos and Boos, 1957). Where the mountains join the Great Plains, the foothills region consists of steeply dipping Paleozoic and Mesozoic sedimentary rocks forming hogback ridges and gravel-covered pediments. The Golden Fault, a high-angled reverse fault, separates the mountains from the plains (Rocky Mountain Association of Geologists, 1972; Figure 3).

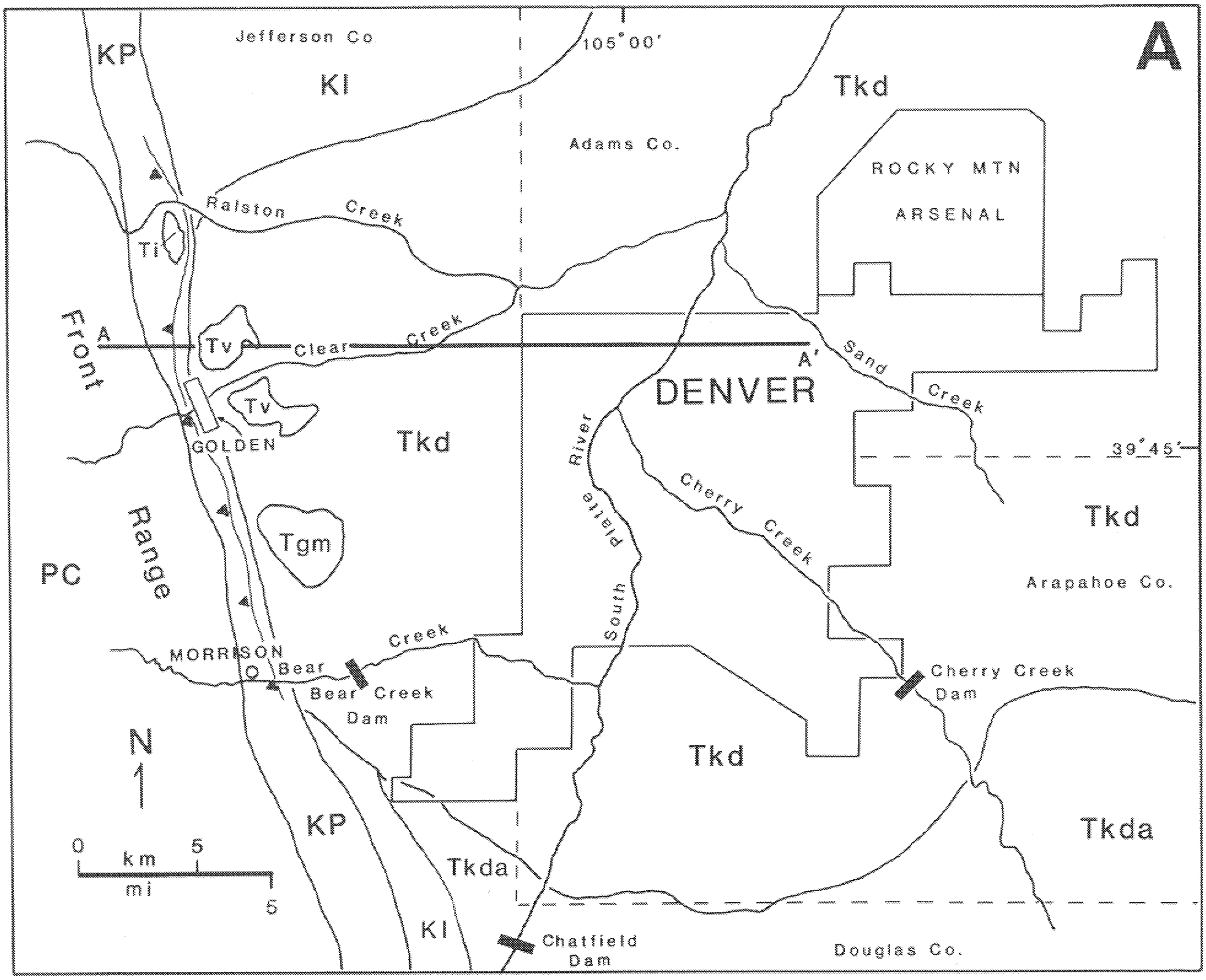
Denver lies near the western edge of one of the largest structural basins in the Rocky Mountain region, the Denver Basin (Figure 4). This basin was formed during the late Cretaceous and early Tertiary time. It is a north-south trending asymmetrical basin with a gentle dipping east flank. The deepest part is under the City of Denver where more than 13,000 ft (3,962 m) of sedimentary rocks ranging in age from Pennsylvanian to Paleocene are present.

Geologic History

Precambrian granites, metamorphosed igneous and sedimentary, and volcanic rocks form the mountains of the Front Range west of Denver. In the foothills, steeply dipping Paleozoic and Mesozoic rocks outcrop and record two invasions of shallow seas. The youngest rocks are of Mesozoic and Tertiary age and indicate volcanic activity in the foothills and in the Front Range (Figure 3). Bedrock stratigraphy and a description of rock units are shown in Figure 5.

At the end of Cretaceous time, uplift in Colorado that built the Rocky Mountains began as part of the Laramide Orogeny. Marine and nonmarine Paleozoic and Mesozoic sedimentary rocks across the uplift were eroded; and along the eastern margin of uplift they were steeply tilted, and some were overturned. As the mountains rose, the land east of the Front Range subsided, forming the Denver Basin. Thus, some rocks that outcrop against the mountains a few miles west of Denver are thousands of feet below the ground surface under the City of Denver (Figure 3B). The Denver Basin was a site of deposition for sediments eroded from the mountains, including the Arapahoe, Denver, and Dawson Formations. Most excavations in bedrock in the City of Denver will encounter the Denver Formation which, under the city, dips gently eastward at angles generally less than ten degrees.

The Upper Cretaceous Arapahoe Formation consists of discontinuous beds of light gray to yellow-brown sandstone and claystone of terrestrial origin. The Denver Formation is highly variable in texture and composition, consisting of light gray to brown tuffaceous silty claystone, tuffaceous arkose, and andesitic conglomerates. These sediments were deposited on a gently sloping surface of low relief in a climate that was warmer and wetter than the present climate (Brown, 1962). The Cretaceous-Tertiary boundary is present in the lower portion of the Denver Formation. Southward, the Denver Formation merges into the Dawson Formation, which is similar to the Denver but is sandier and contains less volcanic materials. Both the Denver and Dawson Formations are loosely consolidated and become finer-grained and thinner eastward away from their source areas. The weathered volcanic material in the Denver Formation commonly swells when wetted, and is thus the cause of a major engineering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Tkd
Tgm
Platte
PC
South
Arapahoe Co.
Creek
MORRISON
Bear
Bear Creek
Dam
Cherry Creek
Dam
N
Tkd
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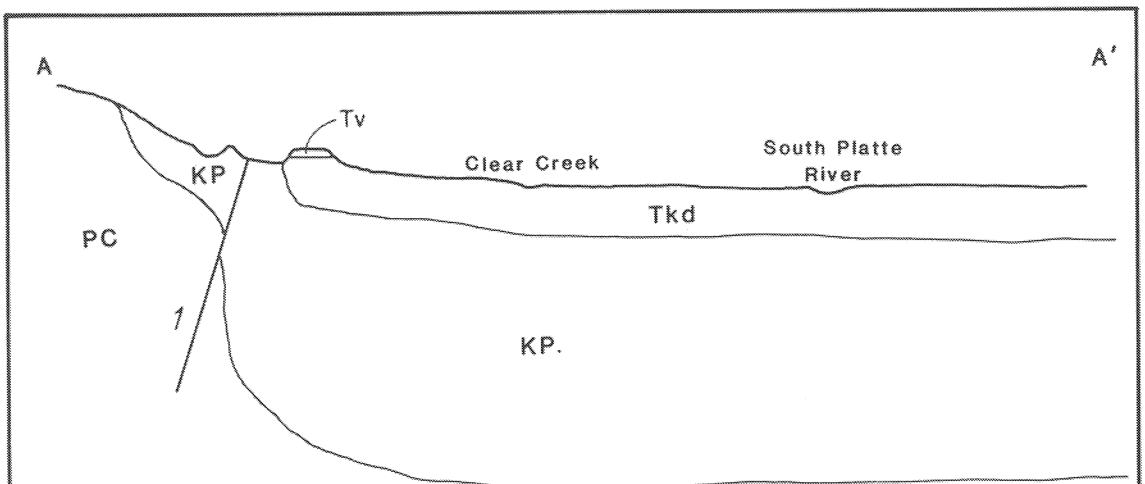
0
km
5
mi
5
Tkda
Kl
Chatfield
Dam
Douglas Co.


A
A'
Tv
KP
Clear Creek
South Platte
River
Tkd
PC
KP.
PC
B

problem in the Denver area—swelling soils. North and east of Golden, potassium-rich basaltic flows are interbedded with rocks of the upper Denver Formation, capping North and South Table Mountains. The lavas flowed southeast about 63 to 64 million years ago from old vents now marked by intrusive outcrops northwest of Denver. The flows are about 240 ft (73 m) in total thickness (Van Horn, 1976).

The Green Mountain Conglomerate consists of a conglomerate, sandstone, siltstone, and claystone deposited as basin-fill material by a through-flowing stream draining from the rising Front Range to the west. The formation is found only on Green Mountain, located southwest of Denver, where it is 600 ft (183 m) thick (Scott, 1972a; Figure 3).

Bedrock in the vicinity of Denver was severely eroded prior to the deposition of overlying unconsolidated Quaternary surficial deposits. Analysis of consolidation tests on samples of Denver Formation from downtown Denver indicate 1,000 to 1,400 ft (305–427 m) of Tertiary material once covered the present bedrock (Committee on Denver Subsoils, 1954). The bedrock surface is very irregular. Numerous paleovalleys filled with unconsolidated Quaternary surficial materials underly the city. Alluvial deposits 100 ft (30 m) thick fill an old paleochannel of Cherry Creek, which trends northward from Cherry Creek Reservoir and joins the South Platte River 9.5 mi (15.3 km) north of the present confluence (Hamilton and Owens, 1972b; Shroba, 1980).

Surficial Deposits

In some parts of the Denver area, bedrock appears at the surface and is covered by thin colluvium and residuum formed by in situ weathering. However, most of the bedrock is covered by alluvial and eolian deposits to depths as great as 100 ft (30 m) (Figure 6). In the downtown area, depth to bedrock averages 20 to 40 ft (6–12 m). The surficial geology of the Denver area was first mapped by Hunt (1954) and was the pioneering work on Quaternary stratigraphy in the Denver area. Quaternary

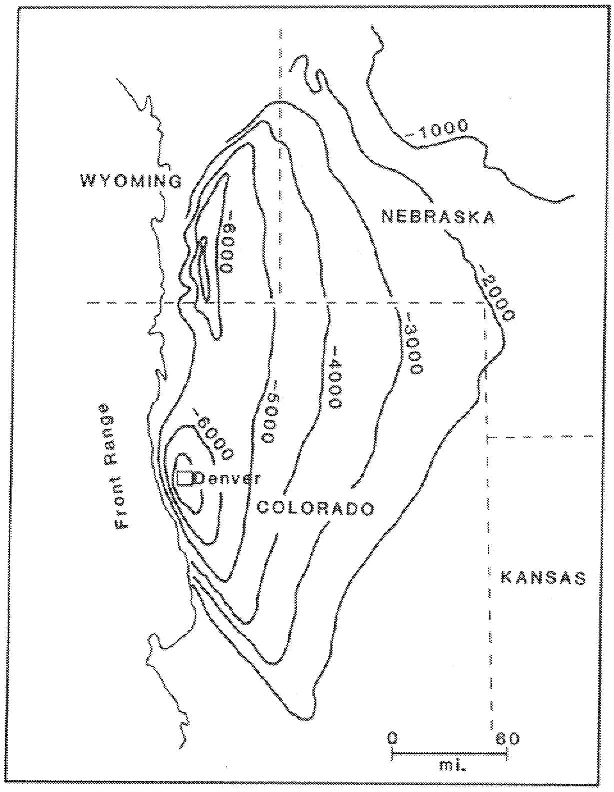


Figure 4. Structure contours on top of Precambrian basement rocks outline the Denver Basin. Contour interval is 1,000 ft (305 m) (from Matuszczak, 1976).

geology and surficial deposits in the Denver area have been further studied by many investigators (e.g., Malde, 1955; Scott, 1960, 1962, 1963; Lindvall, 1978, 1979a, 1979b, 1980; Shroba, 1980; and Van Horn, 1976). Quaternary stratigraphy of the Denver area is shown in Figure 7.

Rocky Flats Alluvium (Scott, 1960) consists of boulders, cobbles, pebbles, and sand in a matrix of locally, red clay. The deposit originated as alluvial fans deposited on pediments by streams draining the Front Range. Thick caliche (pedogenic calcium carbonate) occurs in the upper part of the unit, where it has not been eroded. This alluvium is relatively old—Nebraskan or Aftonian in age (Scott,

←

Figure 3. Generalized bedrock geology of the Denver area (A): Map (modified from Emmons et al., 1896; and Trimble and Machette, 1979) Tgm = Green Mountain conglomerate; Ti = intrusive monzonite; Tv = potassium-rich basalt; Kl = Laramie Formation; Tkd = Denver/Arapahoe Formations; Tkda = Dawson/Arapahoe Formations; KP = Pennsylvanian through upper Cretaceous sedimentary rocks; PC = Precambrian cystallines teeth on upthrown side of Golden Fault; A–A′ = Line of cross-section shown in Figure 3B. (B): Schematic cross-section (not to scale) along A–A′ (Modified from King, 1969).

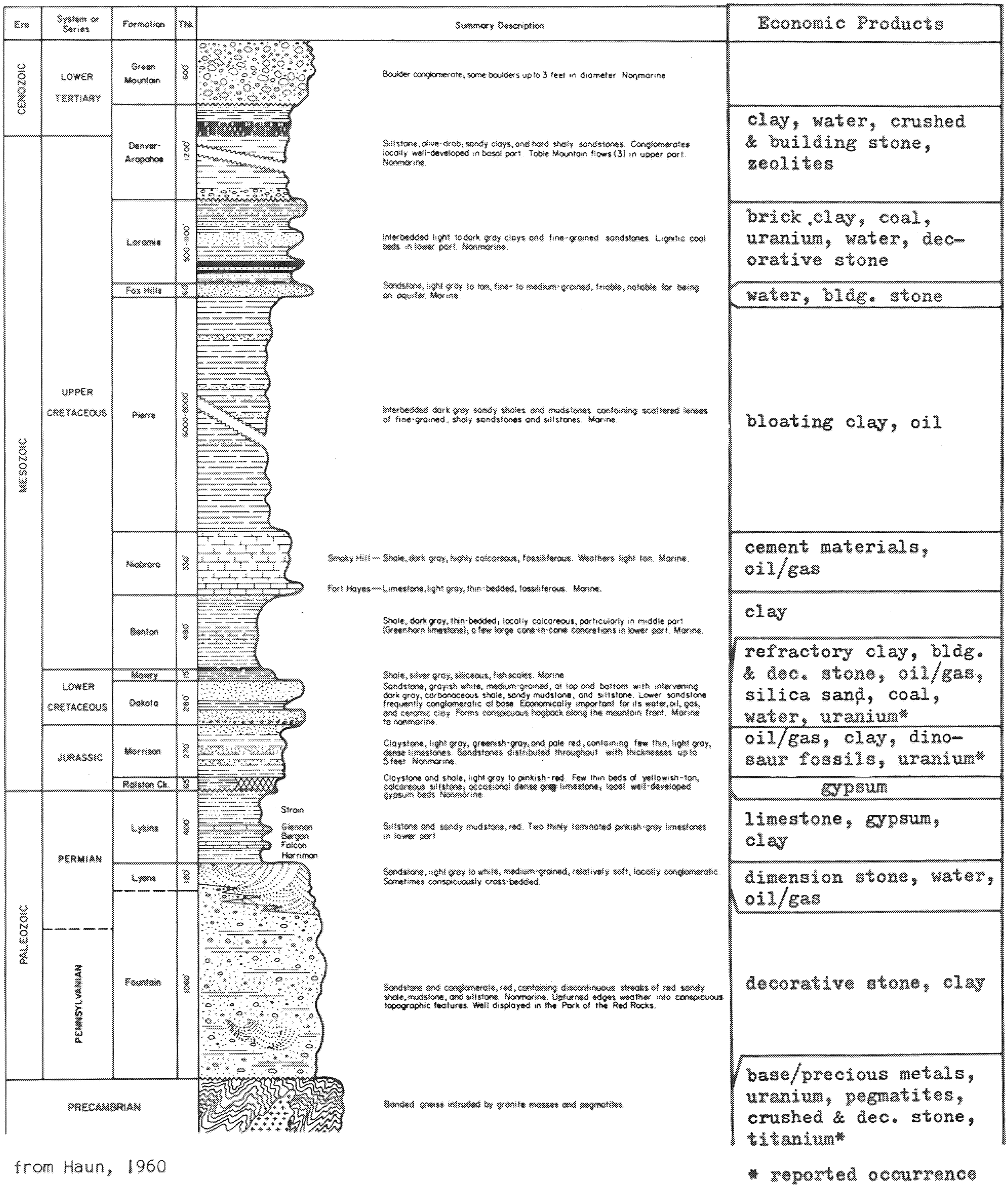


Figure 5. Bedrock stratigraphy, and economic products at Golden, Denver metropolitan area.

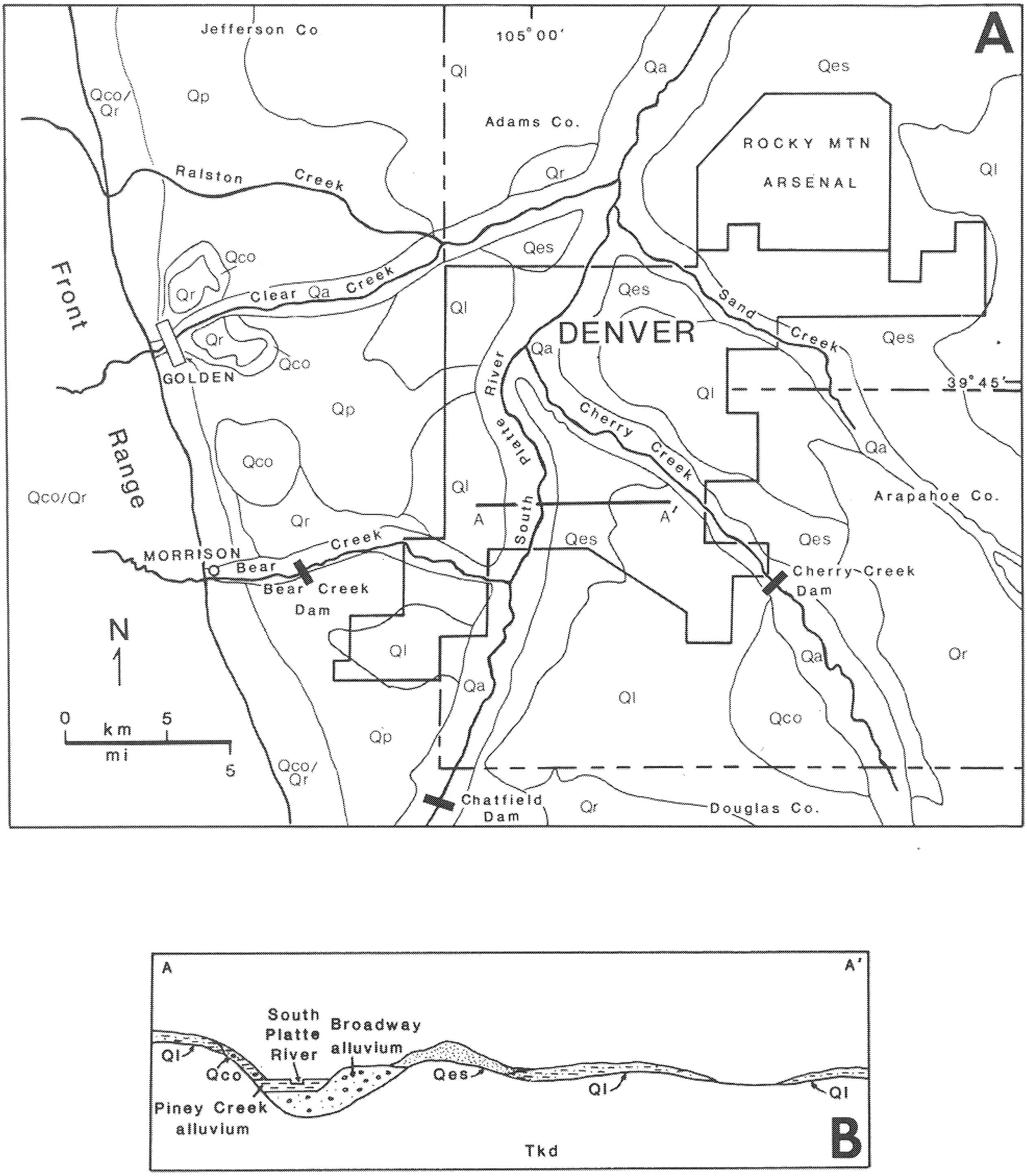


Figure 6. Generalized surficial geology of the Denver area (A): Map (modified from Hunt, 1954; Chase and McConaghy, 1972; and Trimble and Machette, 1979). Qa = alluvium (includes Post-Piney Creek, Piney Creek, Pre-Piney Creek, Broadway, and Louviers Alluviums); Qco = colluvium (includes some landslide deposits); Qes = eolian sand; Ql = loess; Qp = pediment alluvium (includes Slocum, Verdos, and Rocky Flats Alluviums); Qr = residuum (includes bare rock areas). Some deposits may be thin and discontinuous. A–A' = Line of cross-section shown in Figure 6A. (B): Schematic cross-section of surficial geology (not to scale) along A–A'.

Period	Epoch	Relative age		Formation or deposit
QUATERNARY	HOLOCENE	Late		artificial fill colluvium Post-Piney Creek alluvium eolian sand/loess Piney Creek alluvium
		Early		colluvium (soil) Pre-Piney Creek alluvium eolian sand/loess
	PLEISTOCENE	Wisconsinan	Pinedale	colluvium Broadway alluvium loess Louviers alluvium (?)
		Sangamon		(soil)
		Illinoian	Bull Lake	Slocum alluvium
		Yarmouth		(soil) xxx(ash)xxx
		Kansan		Verdos alluvium
		Aftonian		(soil)
		Nebraskan		Rocky Flats alluvium

Figure 7. Quaternary stratigraphy of the Denver area (compiled from Hunt, 1954; Scott, 1960, 1963; Pierce et al., 1976).

1960); and thus, many of the gravels are highly weathered. The exception is a large deposit of Rocky Flats Alluvium northwest of Denver where 80 percent of the gravels are hard quartzite. A very strongly developed soil occurs in the upper part of the Rocky Flats Alluvium. The presence of this, and other paleosols, is important since they can act as compressible clay layers which could adversely affect foundation stability. Rocky Flats Alluvium averages 15 ft (4.6 m) in thickness and occurs on gently sloping uplands 350 ft (107 m) above present streams. The most extensive deposits are northwest of Denver, but Rocky Flats Alluvium is also found both north and southeast of the city (Trimble and Machette, 1979).

Verdos Alluvium (Scott, 1960) consists of brown, well-stratified boulders, cobbles, and coarse sands weakly cemented by clay and calcium carbonate. Many gravel clasts are weathered and crumble when handled. The alluvium was deposited as terrace fills and as mantles on pediments by streams flowing eastward from the Front Range. A distinctive bed of volcanic ash is found near the base of Verdos Alluvium in about a dozen locations in the Denver area. This ash is equivalent to the Pearlette Ash of Kansas and Nebraska and thus is about 600,000 yrs old. Verdos Alluvium is therefore considered to be Kansan or Yarmouth age (Scott, 1963). A very strongly developed soil is also found on top of this deposit and is probably of Yarmouth age. Verdos Alluvium averages 15 ft (4.6 m) in thickness and is found 200 to 250 ft (61–76 m) above present streams. The most extensive deposits are found in west and southwest Denver, west of the South Platte River (Trimble and Machette, 1979).

Slocum Alluvium (Scott, 1960) is a moderate reddish-brown, well-stratified cobble gravel and clayey coarse sand containing abundant mica. Upper layers can be weakly cemented by calcium carbonate. Many gravel clasts are rotten, and the Slocum deposits are distinctly finer grained than older alluvium. The Slocum Alluvium was deposited by streams flowing eastward from the Front Range (Scott, 1963). The Alluvium is believed to be of late Illinoian or early Sangamon age. A very strongly developed soil is found on the top of this alluvium. The deposits average 25 ft (7.6 m) in thickness and are found 80 to 118 ft (24–36 m) above present streams. In the Denver area, Slocum Alluvium is found north and northwest of the city, and in the southwest along the south side of Bear Creek Valley (Trimble and Machette, 1979; Lindvall, 1979b).

Louviers Alluvium (Scott, 1960) is red to yellow-brown, medium- to coarse-grained, pebbly and cobbly arkosic, well-stratified sand and gravel. Small amounts of calcium carbonate can be found locally in the upper layers. The alluvium originated as stream deposits in previously eroded valleys draining the Front Range. Louviers Alluvium is of early Wisconsinan age; and, unlike pre-Wisconsinan gravels in the Denver area, the gravels are not highly weathered. In fact, Louviers Alluvium is the major source of commercial sand and gravel in the Denver area. Alluvial thickness is highly variable, ranging from six to 100 ft (2–30 m). Louviers Alluvium forms terraces 65 ft (20 m) above present streams and locally extends as much as 30 ft (9 m) below present stream level. The alluvium is found mostly northwest, west, and east of the center of Denver (Trimble and Machette, 1979).

Broadway Alluvium (Hunt, 1954) is reddish-brown, fine- to coarse-grained sand and pebbles. The gravels are generally less than one in. (2.5 cm) in diameter; and thus, the Broadway Alluvium is distinctly finer than Louviers. Broadway Alluvium forms terraces 25 to 40 ft (7.6–12 m) above present streams. In the Denver area, the alluvium forms a pronounced terrace along the east side of the South Platte River through the city. The largest and tallest buildings of downtown Denver are built on the Broadway terrace. Broadway Alluvium is late Wisconsinan (Pinedale) in age and is usually less than 30 ft (9 m) thick.

Loess, consisting of silt with smaller amounts of clay and sand deposited by the wind, is generally found downwind from areas of eolian sand (Figure 6). In south Denver, this boundary lies on the western edge of the University of Denver campus (Hunt, 1954; Shroba, 1980) (Figure 6). Loess is the single most extensive surficial deposit in Denver (Committee on Denver Subsoils, 1954; Trimble and Machette, 1979). Loess and eolian sand underlie an estimated 60 percent of the City of Denver. The loess ranges in age from late Pleistocene to early Holocene.

Eolian sand consists of well-rounded, very fine- to medium-grained sand and sandy silt, derived mainly by wind erosion from both old and young alluvium in stream valleys. It covers most of the uplands east and southeast of the major valleys; but the deposits thin toward the south and southeast. The eolian sand is believed to be early-late Holocene in age (Scott, 1963) and generally extends for one to two mi (1.6–3.2 km) downwind of the source areas.

Pre-Piney Creek Alluvium is light brown to yellow-brown, well-stratified, pebbly silt and sand. It is found 15 to 20 ft (4.6–6 m) above present streams in localized sites along small tributaries in the Denver area. Pre-Piney Creek Alluvium has a moderately strongly developed soil, and deposits have been radiocarbon-dated as approximately 5,500 C-14 yrs old (Scott, 1963).

Piney Creek Alluvium (Hunt, 1954) is common in nearly every valley in the Denver area. It is a brownish gray, humus-rich, well-stratified silt, sand, and clay. Piney Creek Alluvium originated by sheet erosion from local soil-covered slopes and averages 10 ft (3 m) thick. Scott (1963) believes this alluvium to be about 2,800 yrs old, based on Carbon-14 dates.

Post-Piney Creek Alluvium is usually grayish-brown, loose humic gravel, sand, silt, and clay forming the lowest terraces and the modern floodplains. It is derived primarily from Piney Creek Alluvium and is found less than 20 ft (6 m) above present stream levels. Thickness is usually less than 20 ft (6 m). No soil has formed on this alluvium, which has been dated archaeologically and by Carbon-14 methods as approximately 1,500 yrs old (Scott, 1963).

Upper Holocene colluvium is deposited on slopes by gravity and sheetwash. Thickness is usually greater than five ft (1.5 m), and physical properties vary widely depending on source areas. In various places around the Denver area, landslides have occurred in bedrock and surficial materials. These mass movements include slumps, flows, and falls and are most widespread on the slopes of North and South Table Mountains, Green Mountain, and steeply dipping sedimentary formations adjacent to the Front Range.

GEOTECHNICAL CHARACTERISTICS

The geotechnical characteristics of overburden materials and underlying bedrock in the Denver metropolitan area can be influential factors in determining site-specific building plans and appropriate foundation types. Due to the variable nature of the soil and rock present (Tables 1 and 2), several methods for determining in situ foundation conditions have been used. Usually an exploration program is conducted to determine the general geology and stratigraphy of the site. Particular attention is focused on identifying potential geologic constraints and suitable foundation-bearing strata. Fortunately, the geologic environment of the Denver region is generally favorable for development of a major urban area. Most of the geologic constraints present, such as expansive clays and settling soils, lend themselves readily to engineering solutions. Laboratory testing of overburden and bedrock materials is conducted to define the physical characteristics, engineering properties, and shear strength parameters of soil and rock units for input into foundation design. Typical foundations used in Denver include: spread footings, bearing walls on grade, pads with grade beams, belled piers (caissons) with grade beams, and post-tensioned slabs. Nearly all foundations are designed to fit site-specific conditions dictated by the geology and soils.

Overburden Material

Alluvium, colluvium, eolian sand, loess, and residuum overlie bedrock in the Denver metropolitan area (Table 1). The sands, silts, clays, gravels, cobbles, and boulders that make up these engineering soils occur both as well-defined layers and as lenses and pockets. The depth of overburden varies from less than a ft (0.3 m) to over 100 ft (30 m). The elevation of the eroded bedrock surface can change

Table 1. *Engineering characteristics of surficial deposits.*[a]

Deposit	Values	Trask Coeff.	Dry Density (lbs/ft³)	Sp. Gr.	Moisture Content (percent)	Ll	Lp	Ip	Activity	PVC (lbs/ft²)	Unconfined Compressive Strength (lbs/ft²)	Unified Soil Class.
Colluvium	Range	2.9–10.2	90–106	2.65–2.70	7–23	22–69	NP–40	NP–30	—	0–3,967	—	CH, SM-SC,
	Mean	4.3	99	2.68	15	44	22	22	—	1,253	—	SC, SM, CL,
	No. of samples	7	8	6	8	9	9	9	—	8	—	MH
Post-Piney Creek Alluvium	Range	3.9–7.3	—	—	—	NP–29	NP–18	NP–11	—	—	—	GW, GP, GM,
	Mean	5.1	—	—	—	15	9	6	—	—	—	SC, CL
	No. of samples	5	—	—	—	2	2	2	—	—	—	
Piney Creek Alluvium	Range	—	—	—	—	33–47	17–23	15–27	—	—	—	CL, SC, GS,
	Mean	14.8	—	2.69	—	42	20	22	—	—	—	SM
	No. of samples	1	—	1	—	7	7	7	—	—	—	
Eolian Sand	Range	—	98–113	2.57–2.65	5–22	NP–39	NP–26	NP–23	—	0	626–940	SC, SP, SM,
	Mean	—	105	2.61	12	26	NP	—	—	0	731	ML
	No. of samples	—	23	2	23	25	25	—	—	23	3	
Loess	Range	—	83–114	2.57–2.77	6–27	22–64	NP–43	NP–35	—	0–3,550	1,670–15,304	CL, ML-CL,
	Mean	3.4	100	2.67	15	41	23	18	0.5	810	6,473[b]	CH
	No. of samples	1	48	2	48	55	55	55	—	61	26	
Broadway Alluvium	Range	5.7–20.0	—	—	—	22–47	NP–27	NP–22	—	—	—	SC, CL, SM,
	Mean	12.2	—	—	—	33	19	14	—	—	—	SP
	No. of samples	7	—	—	—	9	9	9	—	—	—	
Louviers Alluvium	Range	1.8–49.0	—	2.65–2.70	—	18–71	NP–38	NP–33	—	400–6,900	—	GW, GP, GM,
	Mean	8.9	—	2.68	—	47	29	18	0.7	2,960	—	SC, SW-SM
	No. of samples	12	—	2	—	7	7	7	—	—	—	
Slocum Alluvium	Range	—	—	—	—	NP–54	NP–28	NP–21	—	900–4,100	—	GW, GP, GM,
	Mean	2.6	—	—	—	31	17	14	0.7	2,430	—	SC, SW
	No. of samples	1	—	—	—	6	6	6	—	—	—	
Verdos Alluvium	Range	—	—	—	—	31–37	22–24	7–15	—	—	—	GP, SC
	Mean	5.6	—	—	—	34	21	13	—	—	—	
	No. of samples	1	—	—	—	2	2	2	—	—	—	
Rocky Flats Alluvium	Range	6.6–>350	—	—	—	33–70	23–33	10–37	—	—	—	GM, GP, GC,
	Mean	107.3	—	2.68	—	54	29	25	—	—	—	SC, CH, MH,
	No. of samples	7	—	1	—	5	5	5	—	—	—	CL

[a] Compiled from: Larsen and Brown, 1971; Van Horn, 1968; Maberry and Lindvall, 1974; Committee on Denver Subsoils, 1954; Shroba, 1980.
[b] 100–500 when wet.

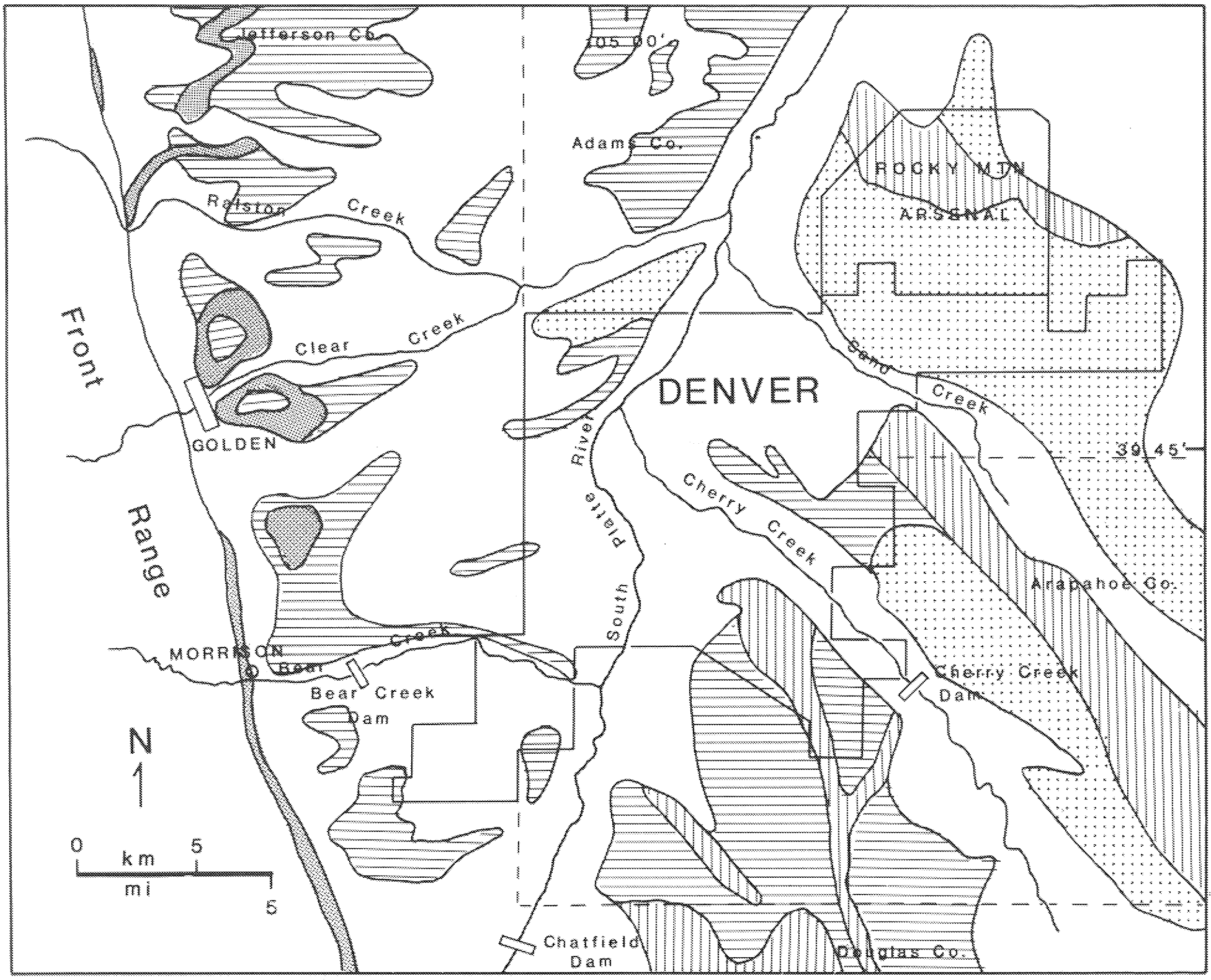
dramatically over short distances. Usually within a given building site, the depth to bedrock is fairly uniform, although changes of up to 20 ft (6 m) have been reported. Subsurface conditions are further complicated by the presence of numerous uncompacted man-made fills.

Shear strength characteristics of the various soil units are determined by composition, thickness,

Table 2. *Engineering characteristics of Denver Formation.*[a,b]

Denver Fm.	Values	Sp. Gr.	Ll	Lp	Ip	Activity[c]	(lbs/ft²) PVC	(lbs/ft²) Unconfined Comp. Stg.	(fps) Vp	(fps) Vs	(lbs/ft³) Dry Density	(%) Moisture Content
Sandstone facies	Range	—	35–74	NP–45	NP–66	—	0–9,900	2,923–972,000	6,400–8,800	2,660–4,900	97–111	4–15
	Mean	2.70	60	28	32	1.03	3,600	302,000	7,690	3,640	106	11
	No. of samples	1	8	8	8	—	9	6	3	3	3	3
Claystone facies	Range	—	41–99	23–59	24–59	—	2,300–19,418	3,341–32,364	—	—	91–114	14–26
	Mean	—	66	35	31	0.65	6,318	12,841	—	—	104	20
	No. of samples	—	32	32	32	—	33	38	—	—	18	18

[a] Compiled from: Van Horn, 1968; Maberry and Lindvall, 1974; Committee on Denver Subsoils, 1954; Shroba, 1980.
[b] Fresh to moderately weathered samples.
[c] Plastic index/percent clay.



Geotechnical Characteristics of Surface Materials

Landslides - potential and active, Expansive soil - high to very high swell potential, Settling or collapsing soil, Sandy soil - potential lateral spreading. Minimal geologic constraints (From Hart, 1974 and Hamilton and Owens, 1972.)

Figure 8. Geotechnical characteristics of surficial material (compiled from Hamilton and Owens, 1972b; Hart, 1974).

density, consolidation and swell properties, and organic content. Related foundation problems can be caused by swelling clays, collapse-prone soils, lateral spreading, subsidence, and mass movements (Figure 8).

Moderately swelling soils are estimated to be present in surficial materials over about 50 percent of the Denver area, particularly in the south, southeast, and western parts as shown in Figure 8. Approximately 25 percent of the area is affected by a high to very high swell potential (Figure 8; Hart, 1974). Swelling soils typically have liquid limits of 45 to 65 percent, and plastic indices of 25 to 35 percent. When tested in a one-dimensional consolidometer, these soils swell 3 to 10 percent under normal loads of 1,000 psf (479 N/m²), but swelling pressures can be as great as 30,000 psf (14,364 N/m²) (Hepworth, 1981). Structural damage can occur when swelling is as little as one percent. Lightly loaded structures supported by shallow foundations, such as single-family houses and highways, are also adversely affected by swelling soils.

Collapse-Prone Soils

Collapse-prone soils are present over approximately 25 percent of the Denver area (Figure 8; Committee on Denver Subsoils, 1954; Hamilton and Owens, 1972b). They consist of low density soils which have high bearing strength when dry, but when moisture is added, lose much of their strength and settle or collapse (Simpson, 1973a). Volume reductions are typically 10 to 15 percent. Some loess and fine-grained colluvium are affected in this manner.

Most Denver-area loess is classified CL in the Unified Soil Classification System and is typically 2 to 15 ft (0.6–4.6 m) thick. Natural water content varies between 6 and 12 percent, and dry unit weight varies between 75 and 95 psf (36–45 N/m^2). The loess generally has adequate dry strength to support structures with foundation pressures up to approximately 3,000 psf (1,436 N/m^2). CL soils (loess) cover about 25 percent of the Denver area (Committee on Denver Subsoils, 1954). Swell-compression tests on Denver loess indicate strengths of 1,000 to 6,000 psf (479–2,873 N/m^2) in its natural dry state, but only 100 to 500 psf (48–239 N/m^2) when moistened. Single-family houses often exert bearing pressures that are sufficient to cause collapse. Dry unit weights of 90 psf (43 N/m^2) or more are generally suitable for single-family houses; dry unit weights of 85 psf (41 N/m^2) or less are indicative of collapse-prone soils (Committee on Denver Subsoils, 1954). Collapse-prone soils are present in the east and southeast portions of the Denver area (Figure 8), in some places overlying swelling soils. However, because the swell potential is usually greater than the settling potential, this geologic constraint is depicted as swelling soils in Figure 8.

Lateral Spreading

Lateral spreading is a phenomena by which foundation support can be lost through the horizontal movement of the foundation-bearing materials. Some portions of eolian sand deposits within the Denver metropolitan region can react to foundation loads in this manner (Maberry, 1972b). Typically, sands affected are clean, well-sorted, and dry. Deposits of this nature cover about 20 percent of the area, generally concentrated in the east and northeast portions (Figure 8).

Compressible Soils

Another surficial deposit that may cause settlement damage to structures is organic silt, defined by Simpson (1973c) as stream-deposited silt that contains more than 10 percent organic material by volume. Carbonaceous matter from the partial decomposition of vegetation has an open structure and will consolidate to a smaller volume by the addition of weight. Organic silts are found in most stream valley floodplains of the Denver area and the underlying low terraces formed of Piney Creek Alluvium.

Consolidation of non-compacted fill in and near old gravel pits has also caused some major foundation problems in the Denver area. Most of the old gravel pits were concentrated adjacent to the South Platte River. The fill was not placed as engineered material. Placement of individual lifts was not controlled, and there was often little or no compaction effort. Significant amounts of organic matter are commonly found in such miscellaneous fills. Due to their organic content, mixed composition, and uncontrolled method of emplacement, most of these old fills are settling, as well as producing explosion-hazardous methane gas. Many developments located on top of uncontrolled fills are also experiencing landscape and structural distress.

Mass Movements

Overburden materials as well as portions of the underlying bedrock can also be affected by soil creep, earth slumps, debris flows, rock falls, and other mass movements. Foundation problems and structural hazards associated with mass earth movements are generally confined to the foothills and the steep slopes of the western and southern sections of the area (Figure 8). Mass movements are discussed further in the section on geologic constraints.

Bedrock Units

The Green Mountain Conglomerate has a very limited extent in the Denver area. It is present only at Green Mountain (Figure 3), and is semilithified and flat lying. It varies from easy to difficult to machine-excavate and is moderately erodible. There are active mass movements on the flanks of Green Mountain; and most of the mountain is considered to have a relatively high landslide potential (Hamilton and Owens, 1972b; Scott, 1972b). Foundation problems encountered within the Green Mountain

Conglomerate relate to the heterogeneous composition of the conglomerate and its potential for instability because of steep slopes.

The Denver Formation underlies most of the metropolitan area east of the foothills. The Arapahoe Formation is found north and west of the city, and the Dawson Formation is located to the southwest. The three formations are similar in overall lithology and engineering characteristics, and are, therefore, not differentiated in this discussion but will be referred to collectively as the Denver Formation.

The Denver Formation is composed of layers and lenses of silty claystone, shale, sandstone, and conglomerate. Numerous silty channel sands occur. Some siltstone and claystone beds contain high proportions of montmorillonite and thus exhibit highly expansive characteristics (Table 2). Siltstone and claystone are usually easy to excavate; cemented sandstone and conglomerate require ripping or local blasting. Where exposed, the Denver Formation is moderately resistant to erosion.

Generally, the Denver Formation provides adequate bearing strength for most structures and is the foundation rock for most of the large buildings in the Central Business District (Table 3; Figure 2). Difficulties occur when the Denver Formation lies at such a depth that interception by drilled piers is economically prohibitive. Associated foundation problems include expansivity of some claystone layers located within the zone of seasonal moisture change, or when the construction process induces increased moisture. Some sandstones filling Cretaceous-aged fluvial channels have proven to be compressible. Denver Formation strata, and some surficial deposits, may contain sulfate salts which have corrosive effects on concrete and metal pipes unless special design procedures are used, such as Type II air-entrained cement and cathodically-protected metal pipes (Hart, 1974; Committee on Denver Subsoils, 1954).

West of the City of Denver, older bedrock units are exposed in relatively thin bands paralleling the Front Range (Figure 5). The Laramie Formation and Fox Hills Sandstone are the first formations encountered. They consist of sandstone, siltstone, and claystone. Economic coal beds are present in the Laramie strata. Rock units are moderately well consolidated to hard. Excavation of claystone and siltstone beds is relatively easy; sandstone is moderately difficult. These formations are also moderately resistant to erosion, but some sandstones can be wind deflated if their surface rinds are disturbed. Slope stability is generally good in unsaturated natural material on slopes up to 25 degrees. Coal zones northwest of the Denver metropolitan area have been extensively mined and some subsidence over mined areas has been reported (Amuedo and Ivey, Inc., 1975). The major foundation problem associated with these bedrock formations are potential expansivity of some claystone layers.

The Pierre Shale contains thin beds of montmorillonite and mixed-order clay minerals, thus exhibiting a moderate to very high swell potential. It is over-consolidated, generally easy to excavate at shallow depth, and only moderately erodible. Slope stability is good where the shale is undisturbed, and in cuts less than 45 degrees where ground water is not present.

The Niobrara Formation (including both the Smoky Hill Shale and the Fort Hays Limestone members) has a very thin outcrop along the foothills on the western edge of the Denver area. It is overconsolidated, moderately easy to excavate, and moderately erodible. Slope stability is generally good on natural slopes where ground water is not present, and in materials of moderate to low swell potential. Few foundation problems are associated with Niobrara strata.

The Benton Shale, which is composed of sandstone, shale, and limestone, is overconsolidated, moderately easy to excavate, and moderately erodible. Slope stability is good on natural slopes up to 45 degrees where ground water is not present. Swell potential is low in sandstones and limestone facies, and moderate to very high in shale facies. Foundation problems associated with this formation are generally related to swell potential.

The Dakota Group consists of interbedded sandstone, siltstone, claystone, and conglomerate. The sandstone is generally hard and very resistant to erosion. It forms the resistant edge of the Dakota hogback present along the foothills west of Denver. The claystone member is soft and rapidly erodible by sheetwash. The group as a whole is difficult to excavate and locally requires blasting. Slope stability is good except along dip slopes where there may be local danger of rockslides where resistant sandstone strata are undercut. Foundation suitability is generally excellent except along dip slopes where the rock may slide.

The remaining bedrock formations have only thin outcroppings in the Denver metropolitan area, and will not be discussed here. Engineering characteristics are discussed in Gardner et al. (1971), Simp-

Table 3. *Major engineering structures.*

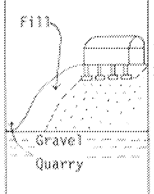
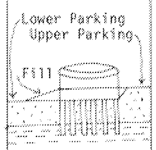
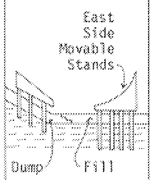
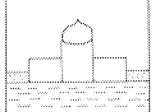
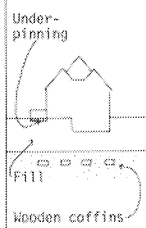
STRUCTURE/ADDRESS	DATE OF CONSTRUCTION	TYPE OF FOUNDATION	GEOLOGIC HOST	PROFILE	COMMENTS	REFERENCE
Denver Coliseum Humboldt Street and Chestnut Place adjacent to I-70	1953	Reinforced concrete pilasters resting on spread footings. Pilasters are 20' long. Footings are 48' long, 7' wide and 6' deep.	Alluvium		This structure is adjacent to a quarry, portions of which have been filled with debris from the stockyards. The building site was moved about 50' east of the quarry to avoid potential settling problems within the fill. A large diameter (2½") drive sampler was developed to test the fill and alluvium.	Peck, 1953.
Denver Sports Arena (McNichols Arena) 17th Avenue and Decatur Street	1974	Caissons-straight shaft 24" to 42" diameter about 20' long 4.0-10.0' into bedrock	Denver Formation		The building is located on sloping terrain. It is partially buried by a planted and landscaped fill which separates the two parking lots.	Plans at City Engineers Office
Mile High Stadium 20th Avenue and Clay Street adjacent to I-25	1948 through 1978	Caissons-straight shaft. 24"-36" diameter; 10-15' into bedrock east side; 5-10' into bedrock west side. (it was critical to keep the east side level due to the movable stands)	Denver Formation		The stadium was originally constructed over the city dump. Portions of the dump have been removed and replaced with engineered fill. The movable stands are the largest structure ever to be moved using water bearings, no settlement has been detected so far.	Plans at Engineers Office, and oral communication with Jim Toole - DMJM, Inc.
Capitol Building Colfax Avenue and Sherman Street	1894	Footing Walls	Denver Formation?		The capitol building is 3 stories high and has one basement. The top of the dome is 272' feet from the ground. This structure was constructed as 3 separate units, the central portion and 2 wings. Each unit was designed to stand independently in case of a partial collapse due to an earthquake. Subsurface access tunnels lead from the capitol building to the adjacent state buildings although they are not used today.	Colorado Dept. of Education Capitol Building pamphlet, and communication with Gretchen Haskins.
Botanical House York Street and 9th Avenue	Around 1900?	Footings	Fill over Alluvium		This historical building began to subside and experience distress. Subsurface investigation revealed an old cemetery under about 20' of fill. The old wooden coffins were collapsing. Several of the gutter downspouts were also discharging water adjacent to the foundation. The downspouts were repaired and the front of the house was underpinned to prevent further distress.	Oral communication with Fu Hua Chen - Chen & Associates, and oral communication with Dr. William Gambill, Botanical House

Table 3. *(continued)*.

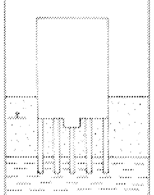
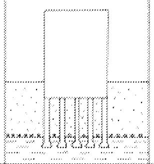
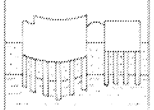
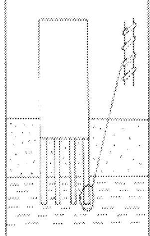
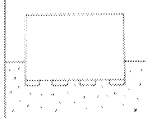
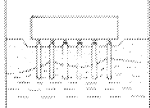
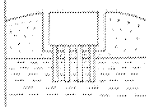
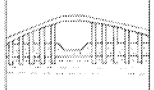
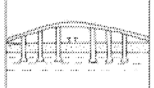
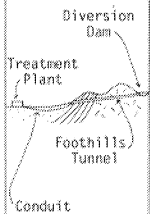
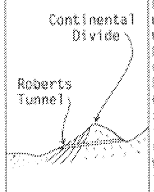
STRUCTURE/ADDRESS	DATE OF CONSTRUCTION	TYPE OF FOUNDATION	GEOLOGIC HOST	PROFILE	COMMENTS	REFERENCE
Police Administration Building 13th Avenue and Cherokee Street	1974	Caissons-straight shaft 70-80' long about 24" diameter 4' to 9' into bedrock (about 60' of alluvium)	Denver Formation		7 story bldg. with 2 sublevels for parking. Lower sections are intermittently dewatered. The lower levels were designed for flooding in the event of a flood on Cherry Creek. The building excavation was made using driven soldier piles and tie backs.	Plans at City Engineers Office and oral communication with Dick Hepworth, Chen and Associates
Lincoln Tower Lincoln Street and 18th Avenue	1961?	Caissons-belled about 70' long about 2.0' into bedrock (about 68 feet of alluvium)	Denver Formation		This building is 13 stories tall with a 2 story subsurface parking garage. Immediately above bedrock a gravel layer was encountered which made drilling the caissions very difficult.	Oral communication: Bob Heister-Architect
Symphony Hall (Boettcher Hall) Curtis Street and 14th Street	1974 to 1979	Caissons-straight shaft 24" to 48" diameter 2' to 9' into bedrock (about 40' of alluvium)	Denver Formation		The hall and the arcade are supported on caissons. The adjacent building is an auditorium which is supported on old timber piles.	Plans at City Engineers Office and oral communication with Dick Hepworth, Chen and Associates
First National Bank 17th Street and Welton Street	1957	Caissons - straight shaft with grooved design 5'6" diameter 15' minimum penetration into bedrock (about 50' of alluvium)	Denver Formation		The First National Bank is a 28 story building with 1 sublevel. This grooved design was common practice around Denver in the 50's and 60's when the drill hole exhibited a very smooth wall. An oversized tooth was attached to the drill bit and used to roughen the wall. It was later discovered the method didn't greatly increase the friction bearing capacity so it has been generally discontinued.	Oral communication and plan inspection with James Tolle, DMJM, Inc.
Denver Public Library 13th Avenue and Acoma Street	1954	Pads 4' x 4' to 8' x 8' ave. 7' x 7' thickness from 1'6" to 2'4"	Broadway Alluvium		The library is a 4 story building with 2 sublevels.	Plans at City Engineers Office
Isaac Newton Junior High School Arapahoe Road and Colorado Boulevard	1962	Caissons - straight shaft 30" diameter 4.0' minimum penetration into bedrock (bedrock is 4-23' below ground surface)	Denver Formation		The caissons heaved due to pressure from expansive soils. They were cut down, the school was releveled and shimmed upon the cut caissons. The cost of the repairs was approximately equal to the original cost of construction.	Oral communication with Fu Hua Chen - Chen and Associates. Also in Chen, 1975.

Table 3. *(continued)*.

STRUCTURE/ADDRESS	DATE OF CONSTRUCTION	TYPE OF FOUNDATION	GEOLOGIC HOST	PROFILE	COMMENTS	REFERENCE
Hillcrest Reservoir Happy Canyon Road and Oxford Avenue	1957	Caissons - straight shaft about 18" in diameter about 22' long about 15' bedrock penetration	Denver Formation		The reservoir began to lose water. Swelling soils apparently caused distress in the joints and seams of the reservoir. Excavating around each pier to reduce the skin friction solved the problem.	Oral communication with Fu Hua Chen - Chen & Associates
15th Street Bridge over the Platte River	1975	Piles 30' long with 8-10' of penetration into bedrock	Denver Formation		The piles could not be driven into the shale bedrock so a hole was first drilled, then the piles were driven in.	Plans at City Engineers Office. Oral communication with Mike Minhos
West Evans Viaduct over Santa Fe RR Tracks	1971	Caissons-belled about 2½" in diameter with 6' minimum penetration into bedrock	Denver Formation			Plans at City Engineers Office. Oral communication with Mike Minhos
Foothills Tunnel	Scheduled for 1982		Greenhorn Limestone Dakota Limestone Fountain Formation Precambrian Metamorphics		Scheduled for completion in 1982. The 3.4 mile tunnel will initially bring 125 million gallons of water per day to the Denver Metropolitan area. The tunnel is capable of conducting 500 millions gallons per day. When used to capacity this will double Denver's existing water supply.	Oral communication with Quentin Hornbeck, geologist, Denver Water Board
Roberts Tunnel	1960		Dakota Formation Benton Shale Niobera Formation Entrada Sandstone Pierre Shale Montazuma Shale Precambrian Metamorphics		Roberts Tunnel is 23.3 miles long and diverts western slope water to Denver by taking water under the Continental Divide. Presently it is operating at 300 to 400 second feet of water but its capacity is 1000 second feet of water. The water is then treated at the Marston treatment plant.	Oral communication with Qunetin Hornbeck, geologist, Denver Water Board.

son and Hart (1980), and McGregor and McDonough (1980).

Exploration and Testing Methods

Exploration and testing methods used to define the surface and subsurface conditions at potential building sites include review of technical literature, surface mapping, and subsurface drilling and/or trenching. Samples are usually taken at regular intervals or at apparent changes of material, and tested to determine their engineering characteristics. Field and laboratory tests are mainly performed in accord with specifications of ASTM (American Society for Testing and Materials) by inhouse labs of private geotechnical firms or by commercial labs on a custom (piece-work) basis. In a few labs, test specifications may differ slightly, or additional non-ASTM tests may be available. Testing is generally performed by trained, supervised technicians.

Laboratory capability ranges from minimal to extensive. The best equipped are the central geotechnical laboratory of the U.S. Bureau of Reclamation, and the rock and soil testing facilities of the Engineering Geology Branch, U.S. Geological Survey in Denver. Geological Survey testing is automated, with test control, data sampling and recording performed by computers. This lab is also developing mobile facilities for appropriate on-site testing, data recording, and radio telemetry of field data (Simpson, 1981).

Some common tests used are Atterberg limits, grain size distribution, dry unit weight, one-dimensional consolidation-swell, and moisture content. The Potential Volume Change (PVC) test was widely used in the past but is not generally used today by Colorado geotechnical engineers (Hart, 1974). The PVC test consisted of a modified floating ring consolidometer in a loading frame with a proving ring. An air-dried, recompacted sample is flooded with water and allowed to swell against the proving ring. After two hours, the moving ring dial is read and converted to a swell index (Hart, 1974).

The primary design tests favored by most local geotechnical engineers for swelling soils are the one-dimensional consolidation-swell test for buildings and the California Bearing Ratio (CBR) swell test for highway subgrades (Hart, 1974; Mock, 1981). Dr. Fu Hua Chen (Chen and Associates) has developed a classification system for swelling soils based on three standard AASHO tests. This system compares the percentage of swell from the consolidation-swell test (1,000 psf (479 N/m^2) surcharge), liquid limit, percentage of the sample finer than the #200 sieve (0.074 mm), and the Standard Penetration Test (SPT) blow count. This system classifies swell as follows (Hart, 1974):

% <#200 Sieve	Liquid Limit	SPT (N) Value	Consolidation Swell (%)	Swell Category
>95	>60	>30	>10	Very high
60–95	40–60	20–30	3–10	High
30–60	30–40	10–20	1–5	Medium
<30	<30	<10	<1	Low

The U.S. Bureau of Reclamation in Denver developed the Holtz-Gibbs classification for swell in the early 1950's. This system compares the plasticity index, shrinkage limit, and the percentage of the sample finer than 0.001 mm to the Bureau of Reclamation swell test at 144 psf (69 N/m^2) surcharge as follows (Hart, 1974):

% <0.001 mm	Plasticity Index (%)	Shrinkage Limit	% Swell	Swell Category
>28	>35	>11	>30	Very high
20–31	25–41	7–12	20–30	High
13–23	15–28	10–16	10–20	Medium
<15	<18	<15	<10	Low

Standard subsurface soil sampling tools used in the Denver metropolitan region are the California (Ring) Sampler, the Standard Split Spoon, the Shelby Tube, and the Continuous Corer. Both the California Sampler and the Standard Split Spoon Sampler are driven into the soil with blows from a 140 lb (64 kg) hammer dropping 30 in. (762 mm). Relatively undisturbed samples two in. (51 mm) in diameter and four to 18 in. (102–457 mm) long can be recovered (Mock, 1981). Larger diameter samples can be recovered by Shelby Tubes and Continuous Coring methods. Soil samples are routinely tested for shear strength, consolidation, and permeability characteristics.

Foundation exploration during the construction of the Denver Coliseum resulted in the development of an early cone penetrometer to test relative densities of coarse alluvial gravels and artificial fill material in a former gravel pit underlying part of the

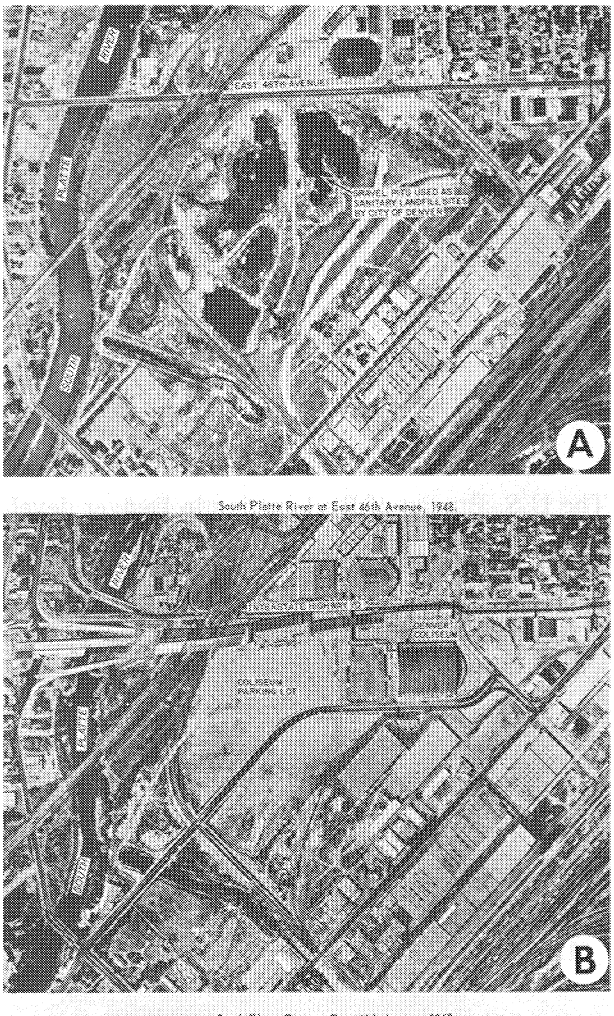


Figure 9. Airphotos of Denver Coliseum construction site, (a) in 1948; (b) in 1965 following filling of gravel pit with municipal waste and subsequent urban land uses (Sheridan, 1967).

building site (Peck, 1953). The base diameter of the penetration cone was 2½ in. (64 mm), and the driving hammer assigned a weight of 350 pounds and a fall of two ft (610 mm). The number of blows required for each ft (0.3 m) of penetration was counted, which led to adequate discrimination of artificial fill material from dense gravel layers. Based on these penetrometer results, the location for the structure was moved 50 ft (15 m) east to allow a spread footing foundation to be located uniformly on the upper surface of an extremely dense part of the gravel deposit (Peck, 1953) (Table 3; Figure 9). This sampler has not seen much local use since its first application.

Foundation Types

Typical foundations used in the Denver metropolitan area are spread footings, bearing walls on grade, pads with grade beams, drilled piers (or caissons) with grade beams, and post-tensioned slabs (Figure 10). The type of foundation depends on the size of the structure, and the surface and subsurface conditions of the site. Spread footings and footing walls are most commonly used for smaller structures such as homes and buildings less than four stories high. High rise structures (more than five stories) are usually supported by drilled piers (or caissons) that are founded in bedrock (Table 3). Drilled piers with grade beams have also been very successful in areas where swelling clay is present. They may be straight shaft, straight shaft with shear rings, or belled.

Building on expansive soils in and around Denver has encouraged the use of various engineering and design treatments. Lightly loaded structures built over soils with low swell potential often use spread footing foundations. With slightly higher swelling potential, footing walls or grade beams supported by pads are utilized (Hart, 1974). Over moderate to highly swelling soils, small-diameter, heavily-loaded, straight-shaft piers are extended to a depth where moisture changes are minimal. The piers are commonly used in conjunction with grade beams. Piers carry structural loads by skin friction along their surface length and by end bearing pressure at its base. Piers are commonly 10 to 20 ft (3–6 m) long and extend three to eight ft (0.9–2.5 m) into firm bedrock. This design is common in Denver because it has been very successful in expansive soils. Many local contracting firms specialize in drilled pier foundations, making this an economical design. Belled pier foundations are not extensively used because the enlarged pier bottom reduces contact bearing pressures on the potentially expansive materials. In the 1950's through the 1960's, shear rings consisting of enlarged zones placed at regular intervals along the piers were used. It was believed that this design increased the friction bearing capacity; however, later tests showed that it usually did not make an appreciable difference; the practice has generally been discontinued.

In highly expansive soils, structural floors are supported with grade beams and piers. A void space is left beneath the floor system to eliminate heaving damage. Edge-stiffened or post-tensioned slabs have been in limited use around Denver. Chemical

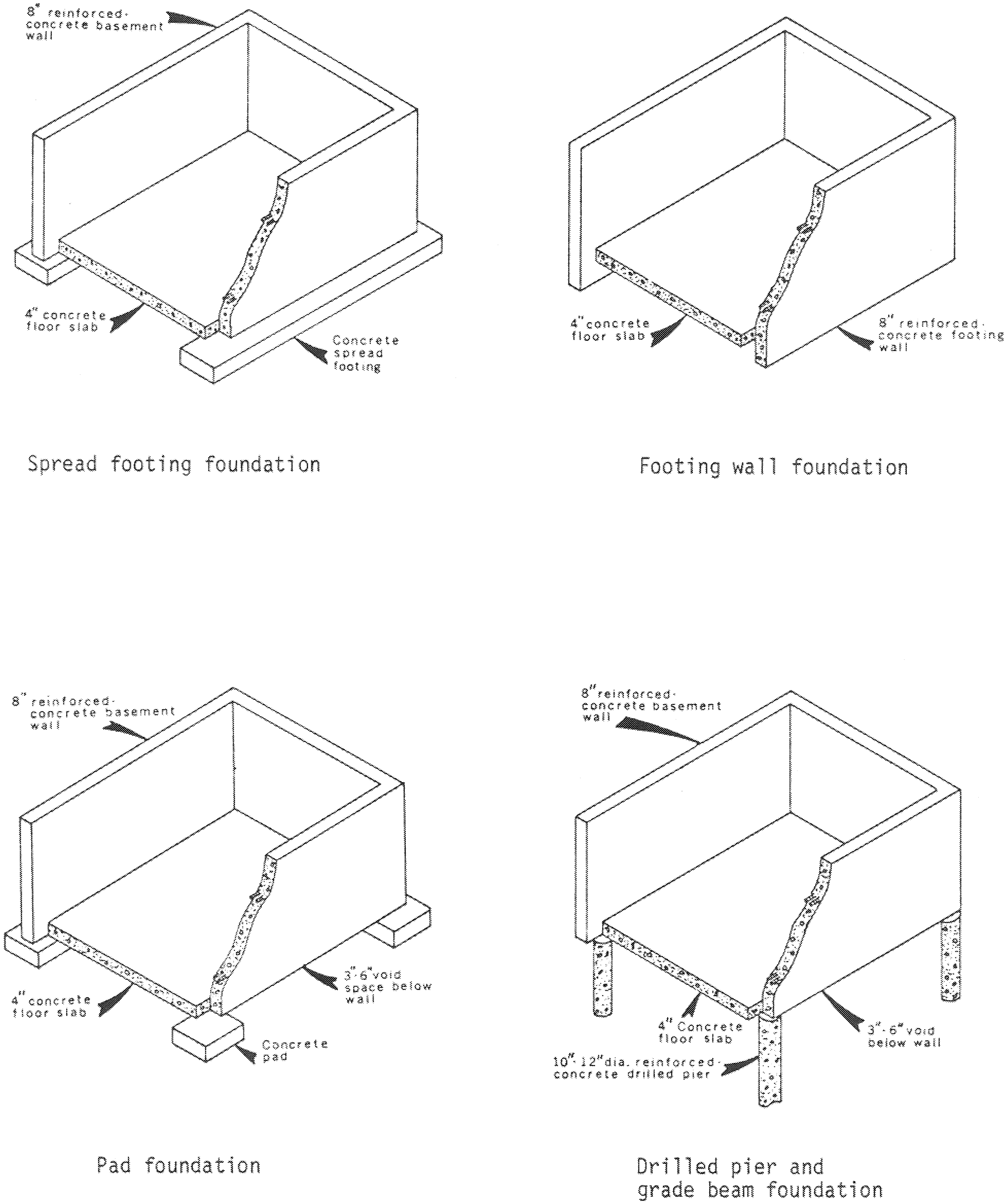


Figure 10. Typical foundations used in the Denver area (Hart, 1974).

stabilization of swelling soils is not as common as special foundation design (Hart, 1974).

Floating slabs are commonly used for on-grade floors. Foundation slab-on-grade construction was used on many post-war houses in Denver through the early 1950's. Costly damage to these houses resulting from expansive soils has virtually halted the use of this foundation type for houses in areas of potential expansivity (Hart, 1974). Presently, slab-on-grade construction is usually confined to driveways, garages, and patios. On moderately swelling soils, slabs can be supported on grade after swelling soils are removed and replaced with non-swelling, impervious soils with low liquid limits.

Homeowners responsibilities in swelling soil areas of Denver include proper drainage and landscaping. Slopes as much as 1:10 (V:H) away from house foundations are in use. Water must not be allowed to pond near foundations, and drain spouts should discharge at least four ft (1.2 m) from buildings.

Highways in the Denver area have been damaged by swelling soils mainly of the A-6 and A-7 AASHO groups, and by borderline soils between the A-4 and the A-6 and A-7 AASHO groups (Lamb and Hanna, 1973). Treatment consists of removing swelling soils and reworking them or replacing them with other selected, nonexpansive fill materials (Hart, 1974). Depth of excavations is determined by plasticity index: 10 to 30, two ft (0.6 m); 30 to 50, three ft (0.9 m); and over 50, four ft (1.2 m) (Sealy, 1973). The use of flexible pavements such as asphalt, rather than concrete rigid pavements, has reduced the cost of repairs.

Several different methods of dealing with collapse-prone soils in Denver have been successful. Impervious foundation and drainage designs have been used to prevent wetting of the soil for the expected lifetime of the structure. Another method used has been to precollapse the soil prior to construction by deep soaking or through the use of vibratory equipment. Some thin deposits are removed prior to construction. The use of drilled pier foundations has been popular in those areas where firm foundation material is present at relatively shallow depths below the collapsible soils.

MATERIALS

Sand and Coarse Aggregate

The Denver metropolitan area was originally endowed with nearly 900 million tons (816 million tonnes) of high quality gravel plus 40 million tons (36 million tonnes) of high quality sand, located in only five percent of the metropolitan area (Inter-County Regional Planning Commission, 1961). In 1935, Denver County supplied 90 percent of its sand and gravel requirements plus those of adjacent Douglas, Adams, and Jefferson Counties. By 1950, Denver's contribution had dropped to only 20 percent. As early as 1957, the Colorado Sand and Gravel Producers Association (1957) predicted that without proper controls, all the available high-quality gravel resources within a 10-mi (16-km) radius of downtown Denver would be depleted by 1977.

High quality sand and gravel in the Denver metropolitan area is restricted largely to floodplains and low terraces of major streams. These deposits are the youngest, least weathered, and least cemented. Rocky Flats, Slocum, and Verdos Alluviums are generally coated with clay and/or calcium carbonate, which inhibit binding with cement and are difficult to remove. These lower quality deposits are found in higher terraces and pediments, can be weathered, and contain an abundance of unsound clasts. Other sand and gravel deposits in the Denver area are found in alluvial fan, pediment, dune, and valley-fill deposits (Figure 11) as well as in floodplains and below stream terraces.

The Colorado Geological Survey has used the following general guidelines for identifying commercial gravel deposits:

—Five-acre (2-ha) tracts with at least 15 ft (4.6 m) of gravel can be considered economic.
—The maximum stripping ratio for commercial valley deposits approaches one unit of overburden for three units of resource (1:3).
—The maximum stripping ratio for terrace and upland deposits can be one to one (1:1).
—Large tracts of high-quality aggregate without overburden may be as little as two ft (0.6 m) thick and still constitute a commercial deposit.
—Commercial gravel deposits should contain a minimum of 30 percent gravel-sized material by weight (Schwochow et al., 1974).

Trimble and Fitch (1974) consider a minimum gravel content of 20 percent of the deposit to be the lower limit under the most adverse foreseeable conditions.

The most significant deposits of commercial gravel are located along the South Platte River and Clear Creek, and in the Rocky Flats alluvial fan located

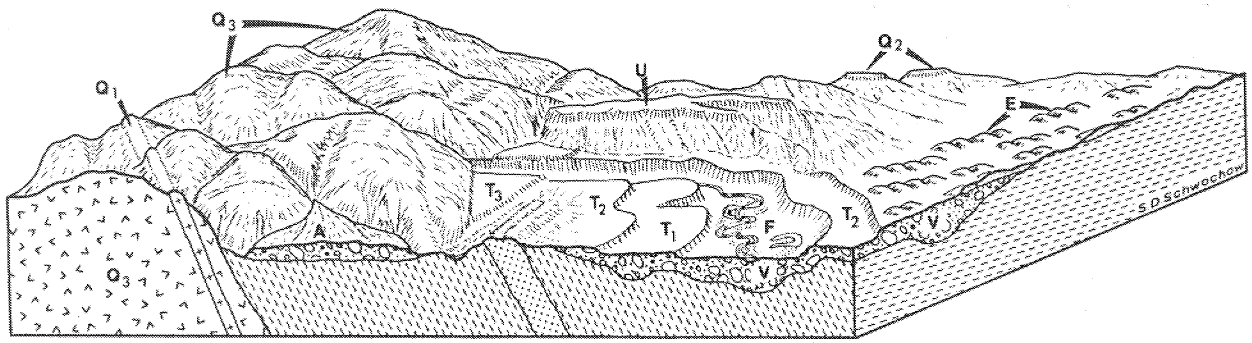


Figure 11. Idealized block diagram showing geomorphic relations among aggregate-bearing landforms. Lowland forms include: valley fill (V), flood plain (F), and terraces (T1—youngest, T3—oldest). Upland forms include gravels (U), alluvial fan (A), and wind-deposited sand (E). Potential quarry-aggregate deposits include fine-grained intrusive igneous rocks (Q1), fine-grained extrusive rocks (Q2), and coarse-grained igneous and metamorphic rocks (Q3) (from Schwochow et al., 1974).

northwest of the city (Figure 12) (Schwochow, 1980). The Rocky Flats alluvial fan contains up to 80 percent quartzite derived from outcrops immediately west of the fan in Coal Creek Canyon. However, this deposit contains a large amount of oversized material. The cobbles and boulders may be suitable for rip-rap, but they are rounded and so may be unstable and hard to place.

Louviers Alluvium is the major source of commercial sand and gravel in Denver, especially along Clear Creek and the South Platte River. Clear Creek Valley contains some of the highest quality gravels available in the Denver area. As a by-product of gravel mining in Clear Creek, operators extract about one ounce of gold for each 1,500 tons (1,361 tonnes) of material processed (Hansen et al., 1976).

Prior to construction of the Cherry Creek Dam and Reservoir in the Southeast Denver metropolitan area (Figure 1), the Bureau of Reclamation conducted extensive tests on three sources of coarse aggregate for concrete appurtenances to the earthfill dam. The coarse aggregates tested included Louviers Alluvium from Clear Creek near Golden, crushed granite from a rock quarry just upstream of Golden on Clear Creek, and crushed basalt from South Table Mountain in Golden (Figure 12). Some test results on these three aggregates are shown in Table 4. Crushed basalt produced a harsh, angular aggregate, and concrete with slightly higher compressive strength, modulus of elasticity, and modulus of rupture. The thermal coefficient of expansion for concrete with basalt aggregate was about eight percent lower than concrete made with the other two aggregates. Concrete made with granite aggregate produced 15 percent less shrinkage on drying than the others. All concretes made with the three aggregates had good resistance to freeze and thaw, produced very good wet and drying durability tests, and had no alkali-aggregate reaction (Hickey, 1950).

The Denver Hilton Hotel in downtown Denver cost $20 million and was completed in 1960 (Figure 1). Its most striking feature is the grill-work of precast concrete framing the exterior walls. All the aggregate used in the facing consisted of pea gravel

Table 4. *Test results of coarse aggregate from the Denver Metropolitan area.*

	Louviers Alluvium (Clear Creek)	Granite (Clear Creek Canyon)	Basalt (South Table Mt.)	Glennon ls (Lykins Fm)	Ft. Hays ls (Niobrara Fm)
Sp. gr.	2.65	2.68[a]	2.73[a]		
24 hr absorp (%)	0.7	0.3[a]	0.9[a]		
Los Angeles abrasion test, %loss, 100 rev.	12.3	8.6[b]	5.9[a]		
% loss 500 rev. (35% loss limit)	41.2[a]	32.2[b]	27.4[a]	44[d]	24.4[d]
Magnesium sulfate soundness test, % loss, 5 cycles (10% loss limit)	9.8[c]	4.8[b]	13.0[b]		

(From Hickey, 1950.)
[a] 2 tests.
[b] 3 tests.
[c] 4 tests.
[d] Van Horn, 1976.

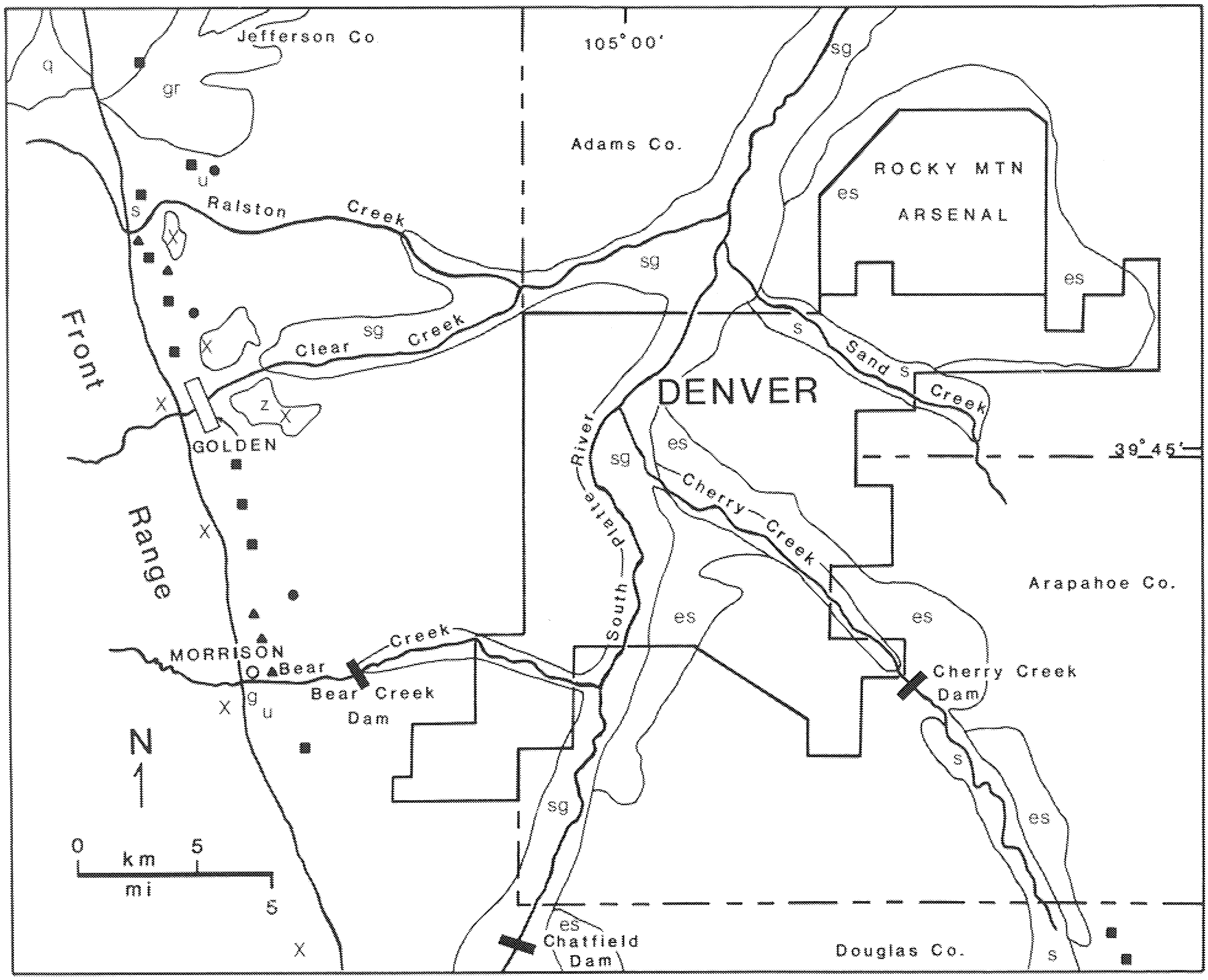


Figure 12. Generalized location of economic mineral resources, Denver metropolitan area. q = quartzite; x = igneous and metamorphic rock quarries; gr = gravel in Rocky Flats alluvial fan; es = eolian sand; sg = sand and gravel; s = silica sand; g = gypsum; z = zeolites; ■ = clay; ▲ = limestone; u = uranium; ● = coal.

excavated and screened from the Broadway Alluvium underlying the construction site. This gravel was used because of its light pink color, soundness, resistance to weathering, and durability. Completed concrete wallings were acid-etched to expose the gravel aggregate (Anonymous, 1959).

Major sand resources occur along Sand and Cherry Creeks and the South Platte River. The material is used for plaster, cement, mortar, blasting, filtration, golf course sand traps, and concrete sand (Schwochow et al., 1974).

The Denver metropolitan area uses more than twice the per capita national average tonnage of sand and gravel, with a value exceeding $22 million per year (Soule, 1974; U.S. Department of Housing and Urban Development, 1978). The metropolitan use represents 41 percent of the state's total sand and gravel production. In 1977, the average price of sand and gravel ($2.00/ton) was double that of 1967, while the price of crushed rock aggregate increased only 44 percent to $2.53/ton (Schwochow, 1980).

Because of the widespread distribution of sand and gravel deposits and the low unit value of the product, industry must be locally oriented in its production and consumption. Unfortunately, many high-quality deposits are now inaccessible in the immediate Denver area because of encroachment by conflicting land uses (Figure 13). Four times as much aggregate has been lost through expansion of

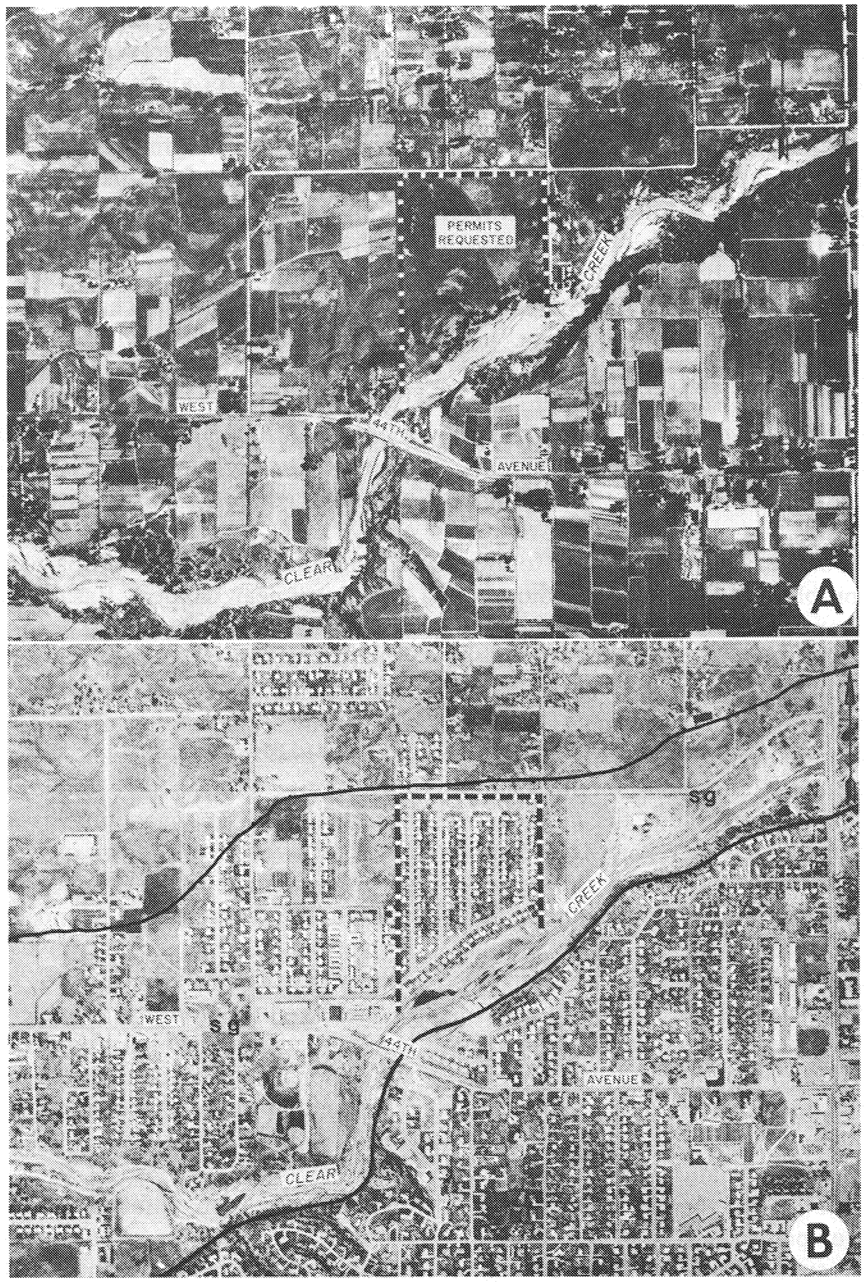


Figure 13. This area (A) is underlain by 20 ft (6 m) of high quality sand and gravel. Three attempts were made in the 1940's to obtain operating permits to mine this land. All requests were denied because of protests from local residents. After the third denial, the land owner sold the property to a housing developer; and as shown (B), the sand and gravel resource has been permanently lost (Sheridan, 1967) (see Figure 1). The principal area of sand and gravel (sg) is outlined in B.

suburbs into areas containing usable deposits than has been consumed in construction (Sheridan, 1967). Well over one-third of the Clear Creek resources have been lost to development in two of Denver's western suburbs, Arvada and Wheatridge. Along Cherry Creek only 10 million tons (9 million tonnes) of the original 30 million tons (27 million tonnes) of sand were mined before encroachment rendered the remaining two-thirds of the resource inaccessible (Inter-County Regional Planning Commission, 1961). The pools of Chatfield and Bear Creek Dams, two recently completed flood-control structures, have precluded the use of enormous quantities of high-quality sand and gravel on the South Platte River and Bear Creek. The remaining large resources of gravel in the Denver area lie to the north of the city in Adams County along the South Platte River from the confluence of Clear Creek north for approximately 11 miles (Figure 12; Schwochow, 1980). These deposits are of lower quality than deposits previously mined in the Denver areas since they contain a smaller percentage of gravel-sized stones, and more sand.

As a consequence of lobbying efforts by sand and gravel producers, in 1973 the State of Colorado enacted legislation requiring that areas of high-quality commercially-extractable deposits be identified and that land use be regulated to insure the protection of this resource.

Local aggregate producers have responded to the resource shortage problem in Denver with four alternatives (Schwochow, 1980). One company began operating a unit train to bring gravel from a pit and loading site on St. Vrain Creek near Lyons, Colorado, about 45 mi (72 km) north of Denver. The 32-car train has a total capacity of 3,200 tons (2,903 tonnes) and hauls gravel to an asphalt mixing plant in Denver. The second alternative is the manufacture of light-weight, expanded aggregate. One company in Denver operated a clay pit in the upper member of the Pierre Shale and an adjacent expansion plant south of Boulder until 1976. Future large-scale expanded aggregate production in the Denver area must wait further evaluation. The third alternative is to mine the lesser-quality sand and gravel in the South Platte River Valley north of Denver. The last alternative is the long-term development of crushed rock aggregate.

The rapidly diminishing sand and gravel resources along rivers in the Denver area indicate that an increasingly larger percentage of coarse aggregate will have to be supplied by crushed rock aggregate (Schwochow, 1980). Rock for crushing in the Denver area consists of Precambrian granite gneisses and schists, quartzite, and Tertiary basalts and monzonite (Figure 12).

Granite gneisses and schists mined for coarse aggregate in the Denver area are all quarried in the Front Range west of the city in Jefferson County. Most crushed rock quarries are located at or near the mouths of canyons because of available transportation routes and proximity to markets. Many are located along major north-west trending faults, long inactive, in the mountain front where brecciation of the rock by faulting has performed the "primary crushing," thus reducing production costs. These quarries produced rock for concrete aggregate, road base, ballast, asphalt binder, and rip-rap. Generally, coarser-grained metamorphic rocks are more satisfactory for crushed-rock aggregate than thinly splitting, highly schistose rocks (Trimble and Fitch, 1974). In the early 1970's, one quarry two mi (3 km) south of Golden provided most of the rip-rap for Chatfield Dam. The quartzite that crops out in a northeast trend about two mi (3 km) wide near the mouth of Coal Creek north of Denver (Figure 12) is one of the best potential sources of high-quality crushed rock aggregate in the Denver area. The quartzite is very hard and crushes to produce clean, angular fragments and very little dust (Schwochow et al., 1974). Basaltic rocks are quarried from extrusive and intrusive outcrops west and northwest of Denver near Golden. Basalt was quarried from South Table Mountain as early as 1905. The crushed rock is used for road materials, concrete and asphalt aggregate, and rip-rap, including that used in the construction of the Cherry Creek Dam (Argall, 1949).

Recently, several large mining operations have been proposed in these igneous and metamorphic rocks. However, the future of crushed-rock aggregate production in the Denver area is uncertain, despite the increasing demand for the resource. Numerous mining permits have been denied by county governments because of concerns of local residents. Alteration and weathering zones, slope stability, esthetic, zoning, and land-use problems are serious limiting factors which will have to be addressed in the instance of future mining.

Clay

Brick and tile manufacturing is one of the oldest industries in Denver. Thomas Warren started the first brickyard in 1859, and by 1860 some brick buildings were in existence; but the majority of structures were wood frame, even though bricks were cheaper than wood (Smiley, 1901). In April of 1863, a fire burned out much of the Denver business area. Seventy buildings were destroyed and scores of residents were homeless. Damages totaled $350,000. A city ordinance subsequently was passed prohibiting construction of wood-frame buildings, and the city rebuilt with brick. The "brick code" in Denver was repealed shortly after World War II for all parts of the city except the downtown core area where wood-frame buildings are still prohibited. Quaternary loess and cohesive alluvium were the major sources of common brick material in early Denver (Ries, 1927).

Refractory and structural clays recently mined in the Denver area are primarily Cretaceous and younger in age, but small amounts were mined from older formations. Clay-producing units have included the Dakota, Benton, Pierre, Laramie, Arapahoe, and Dawson Formations (Crosby, 1977). The Dakota Formation is the sole source of refractory clay in the Denver area, but today, most of the best quality and easily mined refractory deposits have been exhausted (Waage, 1952, 1961).

Demand for refractory clays began in the 1860's with the construction of the first smelters to process ores from the mountains west of Denver (Yingst, 1961). It is ironic that early ore smelters in Golden were located within sight of hogbacks containing high-quality refractory clays, while refractory brick for the smelters was being imported from Wales (Lakes, 1909). However, today refractory clays in the Denver area cannot economically compete with out-of-state clays (Crosby, 1977).

Nearly all clay produced in the Denver area goes to structural clay products such as bricks, tiles, flue liners, flower pots, low-grade ceramics, and sewer pipes. The major non-refractory clay source in the Denver area is the Laramie Formation. Some Laramie clay pits south of Golden have been used for landfill sites (Van Horn, 1976). Structural clays from the Dawson Formation south of Denver are sufficiently valuable that economic hauling distance for this clay to Denver plants is greater than other clays mined in the area (Crosby, 1977; Figure 12).

Several hundred thousand tons of the upper Pierre Shale northwest of Denver were mined yearly between 1961 and 1976 for expanded aggregate. The shale bloated two to three times its original volume when crushed, and rapidly heated to 1,800 to 2,200°F (U.S. Geological Survey, 1968). It had a density of 30 to 60 lbs/ft^3 (481–961 kg/m^3) and had good structural strength and thermal- and acoustical-insulating properties.

Table 5. *Examples of some building stones in the Denver area.*[a]

Stone	Structure
Castle Rock Rhyolite, Douglas Co.	Union Depot, Denver St. Elizabeth Church, Denver Trinity Church, Denver Old Republican Building, Denver Old Board of Trade Building, Denver Old City Hall, Denver University Hall (Old Main), University of Denver
Yule Marble, Gunnison Co.	State Capitol Building, Denver Main Post Office, Denver Federal Reserve Bank, Denver U.S. Customs House, Denver Colorado State Capitol Annex
Salida Travertine, Chaffee Co.	Interior of City and County Building, Denver Denver National Bank, Denver
Wellersville Travertine, Fremont Co.	Denver General Hospital, Denver Gates Rubber Company, Denver
Cotopaxi Granite, Fremont Co.	Base of City and County Building, Denver State Office Building, Denver
Gunnison ("Aberdeen") Granite, Gunnison Co.	State Capitol Building, Denver State Museum Building, Denver
Platte Canyon Granite, Jefferson Co.	Equitable Building, Denver
Masonville Granite Larimer Co.	U.S. Mint Building, Denver
Lyons Sandstone Boulder Co.	Boston Building, Denver Masonic Temple Building, Denver Central Presbyterian Church, Denver
Arizona sandstone	Brown Palace Hotel, Denver

[a] Compiled from: U.S. Geological Survey, 1968; Argall, 1949; Harvey, 1946; Smiley, 1901; Sharps, 1963.

Building Stone

Dimension stone was originally used in Denver pioneer days as foundation blocks to support the full weight of structures. Today, most use in Denver is as decorative veneers, monuments, paving blocks, flagging, curbing, landscaping, and window sills. Among the most popular building stones that have been used in the Denver area are Lyons Sandstone whose slabs are quarried for flagstone curbs, and veneer strips for building facings; Dakota Sandstone used for dimension stone and lichen-covered landscape rock; Yule Marble from the western slope of Colorado for dimension stone, floorings, and steps; and Castle Rock Rhyolite used as rough-dressed dimension stone, window sills, door arches, and garden walls. Table 5 lists some prominent buildings in Denver and the kind and source of stone used in their construction.

The State Capitol building in downtown Denver is an interesting study of the use of a variety of native Colorado building stones. In 1889, gray granite from Gunnison, Colorado, was selected over seven other proposed granites as the building stone for the State Capitol building. The Gunnison Granite was named "Aberdeen" after the famous Scottish quarry (Hunter, 1914). The rock was to be supplied free, resulting in a savings of \$0.5 million over the original design calling for sandstone (Moore and Borland, 1947). The Denver and Rio Grande Railroad built six mi (10 km) of track from Gunnison to the quarry at the expense of the railroad (Moore and Borland, 1947), and the stone was then shipped by rail. The granite quarry employed 150 stone cutters, with an equal number of masons working in Denver. About 280,000 ft^3 (7,930 m^3) of rock was quarried for the capitol building, and the Gunnison ("Aberdeen") Granite blocks were selected to be used for the drilling contest during the 1891 mining congress in Denver. Work on the capitol was interrupted temporarily in 1891 by a strike by quarrymen demanding a nine-hour work day and Sundays off, rather than the customary 10-hour, seven-day shifts of the time (Moore and Borland, 1947). The state capitol building was completed in 1894 at a cost of \$3.4 million. The "Aberdeen" quarry was reopened a year later to supply stone for the exterior steps of the capitol building, and again in 1911–1912 to supply stone for the State Museum Building in Denver.

Yule Marble was used for the State Capitol Annex, as well as the interior stairs and floors of the capitol building. This was done with native marble even though Italian marble could be bought and shipped to Colorado more cheaply than Yule marble could be mined and transported to Denver from the mountains (Colorado Department of Education, 1979). Colorado rose onyx from Beulah (Pueblo County) was used as wainscoating throughout the interior of the capitol. This onyx took seven years to install and cost \$120,000 (Colorado Department of Education, 1979).

Limestone, Silica Sand, Gypsum, Zeolites, Organic Soils

Small amounts of limestone have been mined from Paleozoic and Mesozoic rocks outcropping in the foothills north, west, and south of Denver. The Glennon Limestone member of the Lykins Formation was mined prior to 1960 for agricultural lime (Scott, 1963). The Fort Hays Limestone member of the Niobrara Formation is nearly pure and has been mined, crushed, and used as smelter flux in foundries in Golden and Denver (Scott, 1962; U.S. Geological Survey, 1968), and as agricultural and mortar lime (Figure 12). Fort Hays rock is mined, along with the Smoky Hills Shale member of the Niobrara Formation, for portland cement near Lyons, Colorado, 45 mi (72 km) northwest of Denver (Crosby, 1977).

Silica sand is used in the Denver area as molding sand for iron foundry works, core sand, glass, and cement manufacture (Scott, 1963; Argall, 1949; Crosby, 1977). The source rocks are the Dakota Group sandstones, Lyons, and Lykins Formation (U.S. Geological Survey, 1968; Figure 12).

The first reported production of gypsum in Colorado was from the Ralston Creek Formation at Morrison, about 10 mi (16 km) southwest of Denver, which was worked before 1875 (U.S. Geological Survey, 1968; Figure 12). Small amounts of gypsum were also mined south of Denver in Douglas County and north of Denver from the Lykins Formation for use in plaster, Portland cement, retardant, and as soil conditioner (Argall, 1949; Williamson, 1963; Crosby, 1977).

Zeolites were reported from the Table Mountains in Golden in 1878 where they occur in cavities and veinlets as granular masses as alteration products of silicate minerals in basalt (Figure 12; Gude, 1980; Emmons et al., 1896). While this is a valuable mineral collecting location, no economic production has occurred there.

Organic soils consist primarily of young surficial deposits, especially Piney Creek, and younger alluvium, with humus-rich A soil horizons. These are used in the Denver area as topsoil for landscaping, and sold for soil conditioning material (Scott, 1963).

Coal, Uranium, Oil, and Gas

The entire City of Denver and its suburbs are underlain by subbituminous coal in Cretaceous rocks that lie at a depth less than 3,000 ft (914 m). Some eastern suburbs, such as Aurora, are underlain by lignite at depths less than 150 ft (46 m) (U.S. Geological Survey and Colorado Geological Survey, 1977). However, urbanization has precluded mining in nearly all areas except near the foothills where deposits are nearest the surface and urban development is minimal. Subbituminous coal was mined from the Laramie Formation west of Denver as late as 1950 (Scott, 1962), but mining has now ceased (Figure 12).

More than 100 uranium claims exist in sedimentary rocks in the foothills west of Denver, ranging in age from late Paleozoic to Cretaceous (Scott, 1963). The highest grade ores are found in Cretaceous sedimentary rocks, primarily the Dakota

Sandstone, about one and one-half mi (2.3 km) east of the Front Range near Morrison and Golden (Figure 12). Ores of 0.19–0.26 percent U_3O_8 have been mined from the Dakota Sandstone (Sims and Sheridan, 1964). Deposits typically are primary in origin, and were precipitated as veins along fault zones by ascending hydrothermal solutions (Sims and Sheridan, 1964). Uranium ores with concentrations of 0.35 percent U_3O_8 have also been found in Laramie deposits of coal and adjoining sandstone and claystone in the old Leyden coal mine northwest of Denver in Jefferson County. This mine has been converted to natural gas storage by the Public Service Company of Colorado.

Significant oil and gas fields occur in Cretaceous rocks just north and east of Denver, but most production occurs outside the Denver metropolitan area (U.S. Geological Survey and Colorado Geological Survey, 1977). Non-producing oil seeps have also been discovered west of Denver in Mesozoic sedimentary rocks and Precambrian igneous and metamorphic rocks (Van Horn, 1976).

GEOLOGIC CONSTRAINTS

The Denver metropolitan area has a number of geologic constraints (Hansen, 1976). One of the most significant, widespread, and costly is swelling soils. One Denver geotechnical engineer estimated that one out of 10 houses in Denver suffers, or will suffer, from swelling soil problems; and, in extreme areas, one of three new houses built will have problems (Chen, 1980). This occurs partly because Denver's recent housing boom has pushed construction into problem areas that a few years ago would not have been used.

Swelling soils are generally caused by expansion due to wetting of certain clay minerals (usually montmorillonite) in dry soils. Semi-arid areas like Denver, with pronounced seasonal variations in soil moisture, usually experience the most severe swelling soil problems provided the proper clay minerals are present. Both shrinkage and swelling can occur with moisture variation, and either can cause damage to streets and structures. In the Denver area, swelling has caused most of the damage (Hart, 1974).

Many parts of the Denver/Arapahoe Formations, Pierre Shale, and some of the surficial deposits derived from them, contain very highly swelling clays and have caused millions of dollars in damages (Figure 8). Few areas within the Denver metropolitan region are completely free from potential swelling, however deposits with high to very high swell potential are of more limited extent (Figure 8).

In 1970, Ridge Home, a state school for the mentally retarded, required \$0.5 million in repairs for cracked walls, floors, ceilings, doors, and window frames, in a building only six yrs old (Figure 1). At Isaac Newton Junior High School (Figure 1), \$1 million was spent on repairs for a building only 12 yrs old (Table 3). This expense was equal to the original construction cost of the building.

Subsidence

Another significant geological problem in the Denver area is land subsidence and methane gas accumulation in and near former sand and gravel pits that were mined out and subsequently used for municipal waste-disposal sites (McBroome and Hansen, 1978). These landfills were then graded and converted to various kinds of urban development. Subsidence also has occurred over old coal and clay mines, and from compaction of loess, eolian sand, and organic silts. Hydrocompaction of loess, lateral spreading of eolian sand, and settlement of organic soils have been already discussed under geotechnical characteristics.

Landfills and Methane Gas

Decomposition of organic matter in landfill sites produces a variety of gases, including methane, which is colorless, odorless, and explosive in concentrations greater than five to 15 percent. In June of 1977, two water line construction workers were killed by a methane explosion during construction of water lines near an old trashfill. Figure 14a shows a 1.6 mi (2.6 km) stretch of the South Platte River floodplain in central Denver. This reach was virtually one continuous sand and gravel mining operation in 1949 (Figure 1). With the completion of mining in this area in the 1950's, the pits were used as landfills by the City of Denver, and filled primarily with residential and commercial wastes. No effort was made to compact the fill, nor to place daily or even less frequent earth covers on the debris. Refuse fill ranged in thickness from 20 to 40 ft (6–12 m); and when filled, a three-ft (1 m) cover of clean earth was placed over the entire landfill area. Figure 14b shows the same area 15 years later in 1965. Many of the landfill sites have been utilized for industrial, commercial, and private buildings. In 1978, methane gas concentrations of as much as 62 percent gas by volume of sample were discovered

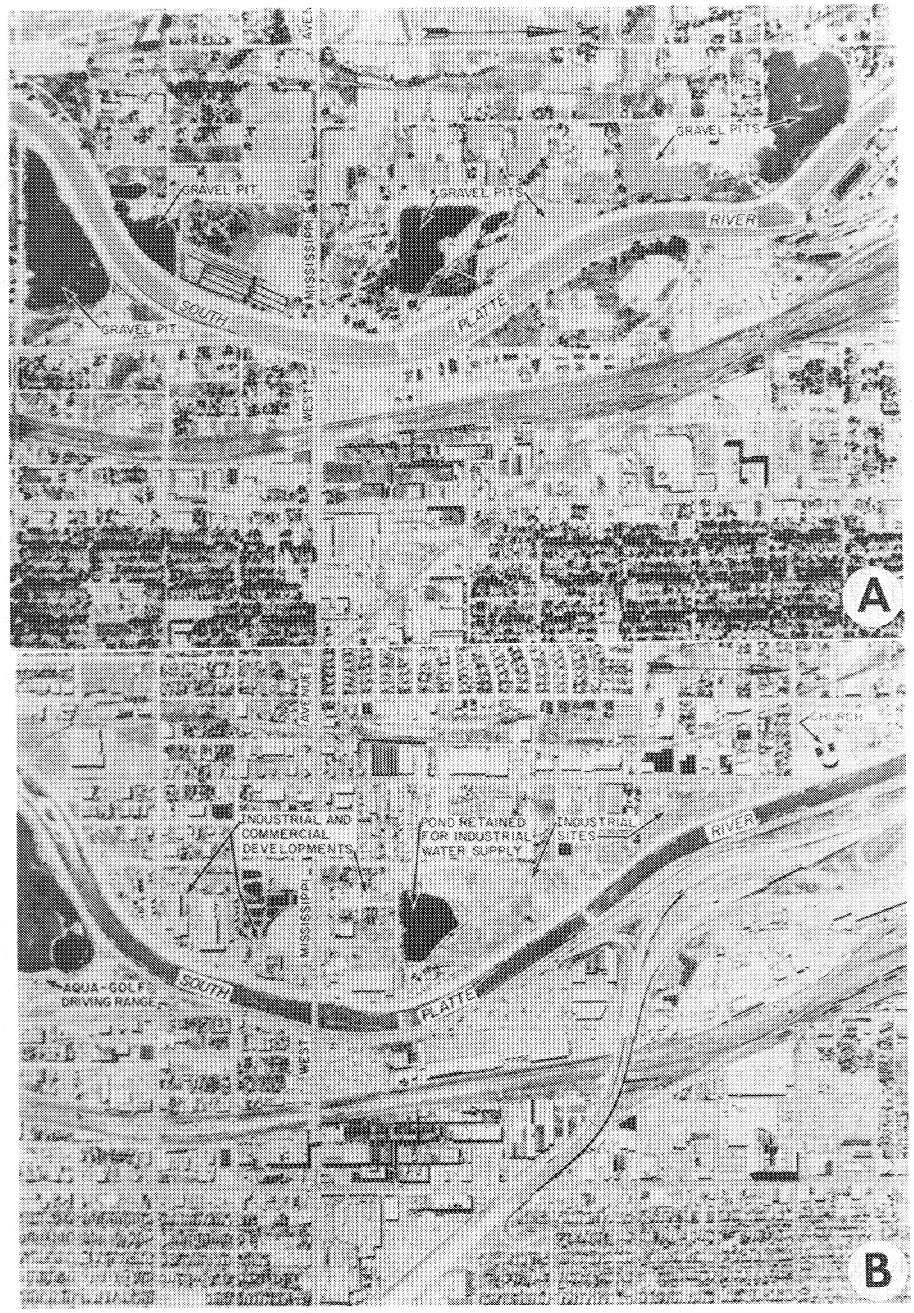


Figure 14. Airphotos of a reach of the South Platte River in Denver, (A) in 1949, and (B) in 1965 following filling of gravel pits with municipal wastes and subsequent urbanization (Sheridan, 1967). The location of the airphotos is shown in Figure 1.

at shallow depths in areas of landfill deposits (Raymond Vail and Associates, Inc., 1979).

One of the highest methane concentrations was found in the church pictured in the upper right-hand corner of Figure 14b. Extensive settlement has resulted in severe damage to basement floors and three ft (1 m) of separation between foundation walls and the basement floor surface. Figure 15 is a photo of some visible exterior subsidence at this structure.

The Cedar Run apartment complex in southeast Denver is built on the site of a sand and gravel pit excavated through Louviers Alluvium in a terrace along the north bank of Cherry Creek (Figure 1).

Between 1950 and 1964, the area served as a landfill for the City of Denver. The property was subsequently purchased and developed into an apartment house complex. The buildings were constructed on caissons extending through the landfill to undisturbed soils below. The structures have experienced only minor distress, but landscaped areas and parking lots exhibit extreme non-uniform settlement. Buried water and sewer lines serving the complex have ruptured on numerous occasions. Structural distress to basement parking lots, pavement slabs, and basement walls has permitted migration of hazardous amounts of methane gas into buildings.

Place Junior High School was originally designed to be located partly on waste fill along Cherry Creek. Foundation engineers recommended the removal of all trash beneath the foundations, and subsequent backfilling with compacted clay soils. Some visible subsidence has since occurred in the street in front of the building, and methane gas has been found in buildings next to the school, but the school itself has been free of foundation and gas problems.

In Denver, design accomodations for structures built over landfills with methane gas problems have included: (a) a constant ventilation system, (b) methane gas alarm systems, and (c) routine inspection of all structures built over former landfills (Raymond Vail and Associates, Inc., 1979).

Clay and Coal Mine Subsidence

Abandoned clay pits have been routinely filled in the Denver region, notably those in the Laramie Formation west of the Colorado School of Mines in Golden. By 1980, extensive infilling with fly ash and flue gas desulfurization waste from the coal-fired Arapahoe generating station in Denver was underway (Figure 16).

Apartment buildings have subsequently been built over some of the older fills. When founded on the sandstone rib walls of the clay pits, the buildings have been little affected by compaction and subsidence. At least one building suffered structural damage when constructed on footings three ft (1 m) deep founded on thick artificial fill overlying a clay pit (Van Horn, 1976). Engineers concluded the foundation failure was caused by the inter-rib earthfill not being properly compacted right next to the ribs because of the two to three ft (0.7–1 m) width over which a sheepsfoot roller cannot reach adjacent to a vertical face. This led to settlement of part of the building foundation which broke sewer pipes.

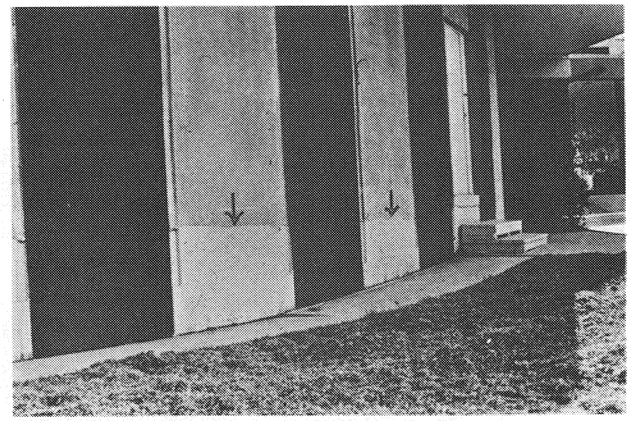


Figure 15. Church shown in Figure 14B showing three ft (1 m) of subsidence of ground surface on outside of building. Original ground surface marked by arrows. Note the temporary stair box below the exit. (Photo taken in April, 1980.)

The broken sewers caused piping of fine sediment from the earthfill down into an old clay-mine access tunnel close below. Enough support was piped out to cause structural failure. The solution was to support building walls with structural steel that extended from one sandstone rib to another, or from one rib to tested compacted earthfill on in situ clay beds that had no adits below (Simpson, 1981).

Numerous underground coal mines in Cretaceous sedimentary rocks west of the City of Denver were worked from the early to mid-1900's. An extensive engineering geologic report on the Boulder-Weld Coal Field northwest of the Denver metropolitan area was produced for the Colorado Geological Survey in 1975 (Amuedo and Ivey, Inc., 1975). This report documented extensive subsidence and property damage over old underground coal mines. In late summer 1981, it was realized that a large shopping center to be built in the southwest Denver area was located over old underground coal mines in the Laramie Formation (Figure 1). A subdivision located over the mines was platted in 1956 and county records show no reference to the coal mines. One contractor who built houses in the subdivision thought major problems with concrete driveways and basements were the result of swelling soils when, in fact, the problems may be related to ground instability from mine collapse (Jenkins, 1981). The existence and location of the coal mines are discussed and clearly located in the USGS report on the geology of the Littleton Quadrangle (Scott, 1962).

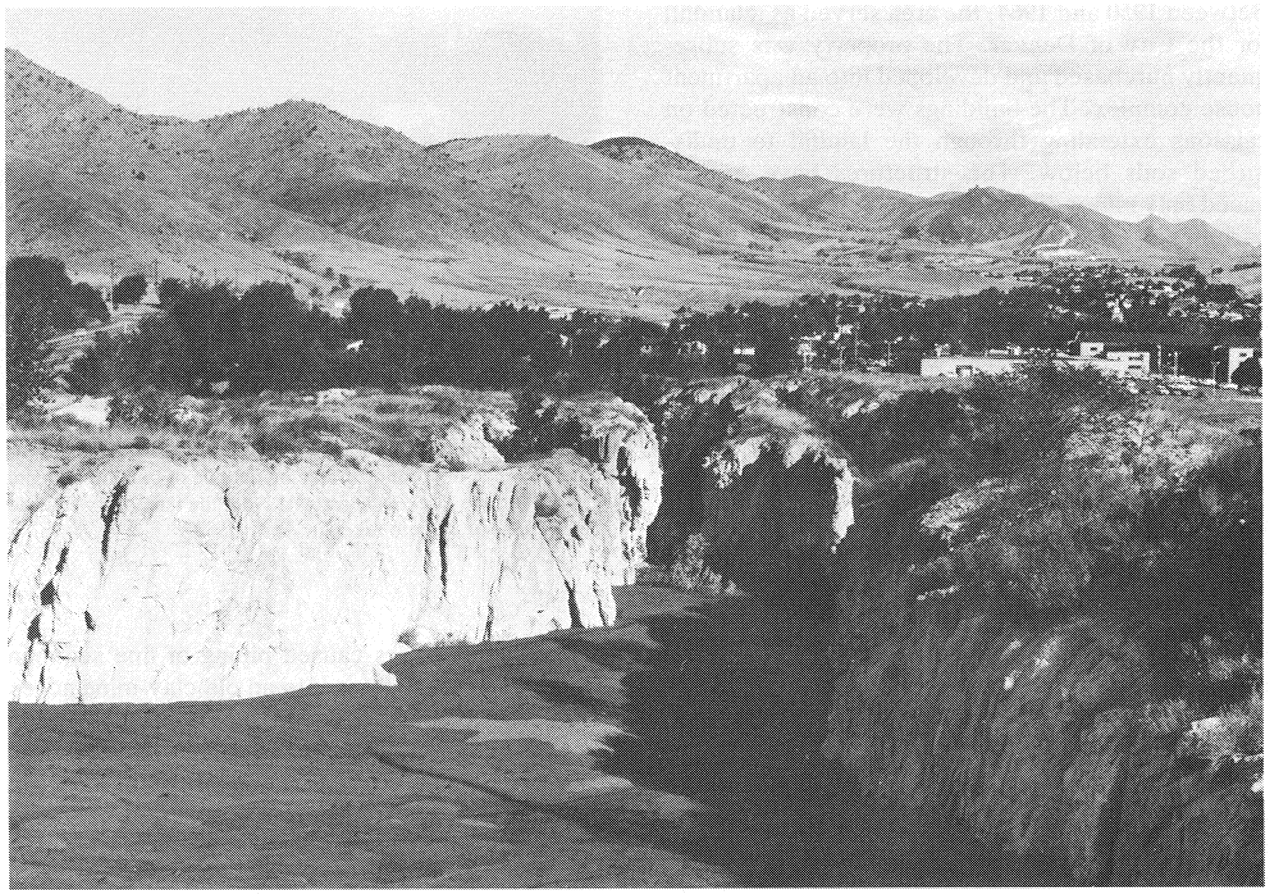


Figure 16. Fly ash slurry disposal as a stabilizing medium for reclamation of abandoned clay pits, Golden, Colorado. The fly ash is delivered by truck, in a wet mixture, over the 15 mi (24 km) distance from the Arapahoe Station of the Public Service Company of Colorado, in Denver. Approximately 3.5×10^3 yd^3 (2.7×10^3 m^3) are produced and delivered each week in a fleet of ten trucks. Mining of the vertically-bedded Laramie Formation clay strata has been going on since about 1876 by members of the Parfet family, who have been managing the fly ash disposal since 1979. Eventual reclamation of the clay pits is planned, with structural foundations designed to span the slurry fills, bearing on the Laramie interbeds (Photograph by Allen W. Hatheway, September 1980).

Mass Movements

A variety of types of mass movements occurs in the Denver area, especially in hilly terrain underlain by Cretaceous and Tertiary fine-grained sedimentary rocks. The probability of slope failure increases significantly where slopes are steeper than 30 percent, and where relief is greater than about 100 to 200 ft (30–61 m). The most significant mass movement hazards exist in the mountains and foothills west of Denver, and south of the city on steep slopes in Douglas County. Steep cut slopes and excavations in some surficial deposits such as eolian sand are also unstable. However, mass movements generally are not a significant problem in the urbanized portion of the South Platte River Valley.

Rockfalls occur south of Denver in the easily eroded shales, mudstones, siltstones, and sandstones of the Dawson Formation, where overlain by a caprock of resistant conglomerate or rhyolite. Rockfalls also occur in the foothills and mountains west of Denver in steep, naturally occurring, or undercut rock slopes of fractured bedrock. Rockslides are found on steep dip slopes of Cretaceous sedimentary rocks in the foothills.

Small earthflows and debris slides also occur south of Denver in areas of steep slopes underlain by clay-rich bedrock and surficial materials (Maberry, 1972a). In the City of Denver, landslides are restricted to clay, silt, and sand-rich colluvial deposits on valley slopes which fail as small rotational slump blocks (Lindvall, 1979b).

Two areas west of the city are the most prone to landsliding in the region—Green Mountain and

North and South Table Mountains. The slopes of Green Mountain are marked by earthflows/debris flows of different ages, presumably post-Pleistocene; some having been active in modern times (Scott, 1972b). The failures on Green Mountain occur in the Green Mountain Conglomerate and underlying Denver Formation.

The flanks of North and South Table Mountains have had a long history of landsliding (Van Horn, 1976). The slopes of the Denver Formation are oversteepened because of a caprock of hard basaltic rock, which results in numerous slumps, rockfalls, and earthflows (Simpson, 1973a, 1973b). Where one road crosses a landslide on the south side of North Table Mountain, the asphalt was estimated to be 13 ft (4 m) thick as successive layers of pavement were added to maintain the road at grade (Conference Field Trip Committee, 1969). A large-area landslide at the north end of North Table Mountain has been estimated to have a total volume of two-thirds to three-fourths mi^3 (2.8–3.1 km^3) (Simpson, 1981).

Conventional methods of mitigating landslide hazards are utilized in the Denver area. These include unloading by grading, drainage provisions, and construction of retaining structures such as buttresses, walls, cast-in-place piles, and tie-back anchors.

Rising Water Tables

In some parts of the Denver area, several hundred homes are plagued by rising ground-water tables and consequent basement flooding. The problem areas are underlain by five to 15 ft (1.5–4.6 m) of permeable surficial deposits above the natural water table, or by an impervious layer of bedrock. The rise in water tables has been attributed to changing of drainage patterns and excessive lawn watering following urbanization. Denver homeowners add an average of 45 in. (1,143 mm) of water to their lawns annually (Shelton and Prouty, 1979), and the recharge to the water table through permeable soils by this method is both rapid, and estimated to be six to seven times more effective than that of natural precipitation (Hamilton and Owens, 1972a). Damages average between $1,000 and $4,000 per affected house. Many such homeowners have had to install sump pumps and shallow dewatering wells.

The basement of the Denver Hilton Hotel in downtown extends 60 ft (18 m) below ground level into the Denver Formation. The ground-water table in the shallow aquifer was just 30 ft (9 m) below street level, and more than two million gallons (7.6 million liters) of water a day was pumped from the foundation site for a period of over two years (Anonymous, 1959).

Flooding

Despite its generally semi-arid nature, an area of the foothills region in Colorado below an altitude of about 7,500 ft (2,286 m) and extending eastward about 50 mi (80 km) onto the Great Plains is subject to very intense cloudburst rainstorms. The usual sources of these cloudbursts are warm, moist Gulf coastal air masses moving northward. Rainfall amounts have been as high as 12 to 14 in. (305–356 mm) in four hours. The magnitude of these storms can be appreciated from the following account of a cloudburst in July of 1896: "The daughter of a rancher was riding on Green Mountain, looking after the stock, when the storm started. By the time she reached the barn, she was practically unconscious on her horse and had to be revived by means used for resuscitating victims of drowning, as the intensity of the rain made it almost impossible for her to breathe." (Follansbee and Sawyer, 1948, p. 22). In a cloudburst in 1921, a horse drowned in an open field (Follansbee and Sawyer, 1948).

The earliest flood in the Denver area occurred in 1844 on the South Platte River. In 1858, Indians told of great floods along Cherry Creek in times past. Figure 17 shows downtown Denver following a flood on Cherry Creek in 1878. We estimate less than 50 people have perished as a result of flooding in Denver since settlement began, but property damage has been very great. Table 6 lists the historic floods of the South Platte River and Cherry Creek. Denver floodplains constitute 30.6 mi^2 (79 km^2) or 10.5 percent of the urbanized area. About 62 percent of this floodplain area has been urbanized (Schneider and Goddard, 1974).

The most disastrous flood in Denver's history occurred on June 16, 1965, when $508 million in damages resulted and six lives were lost. More than 12 to 14 in. (305–356 mm) of rain fell in about four hours in an area south of Denver. Plum Creek, a tributary to the South Platte River draining 302 mi^2 (782 km^2), crested at 154,000 cfs (4,361 cms). The previous maximum known flood was 7,700 cfs (218 cms) in 1945 (Matthai, 1969). The flood peak took two and one-half hours to travel 15 mi (24 km) to the gaging station on the South Platte River at Littleton, where channel and valley storage reduced the crest to 110,000 cfs (3,115 cms). The flood then

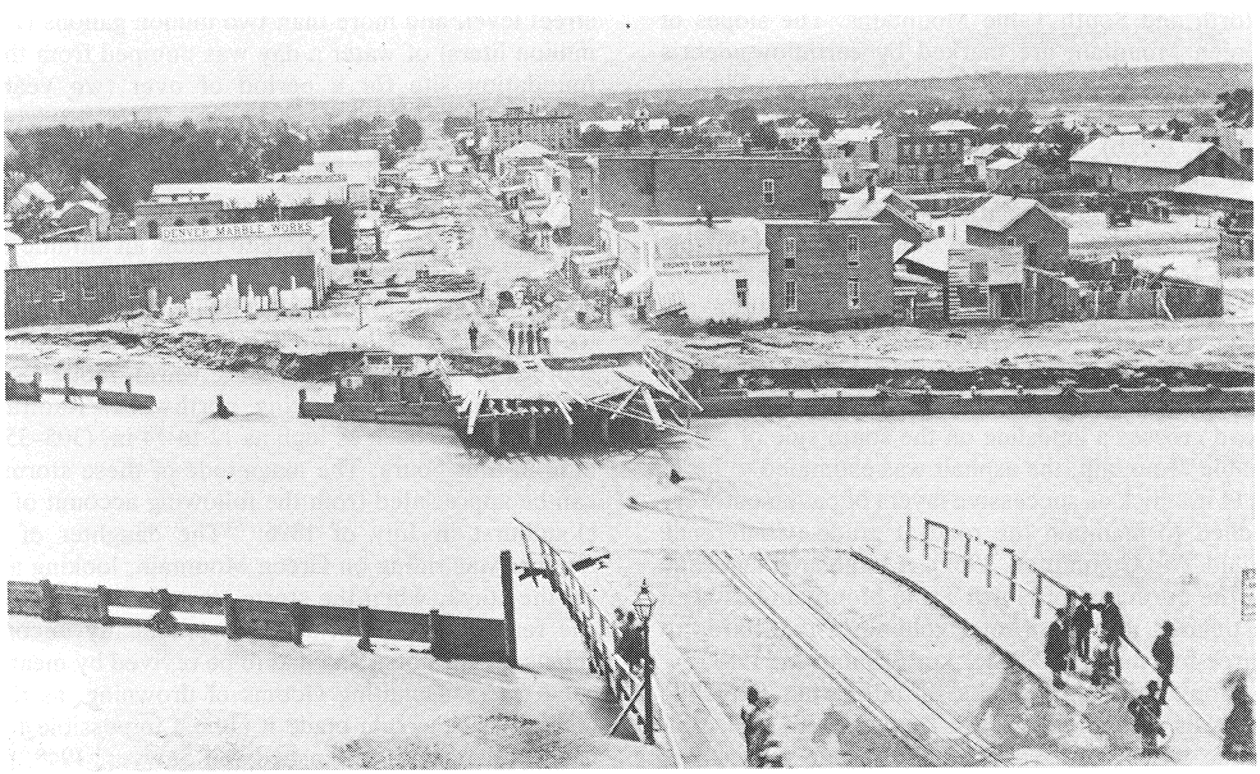


Figure 17. Photograph of Larimer Street bridge in downtown Denver, looking southwest, following flood of 1878. (Photo courtesy of Colorado State Historical Society.)

continued toward Denver, and four and three-quarters hours later the flood had traveled 11 mi (18 km) to the gaging station just below the juncture with Cherry Creek (Figure 1) where the attenuated peak discharge crested at 18.66 ft (5.69 m) and 40,300 cfs (1,141 cms) (Matthai, 1969).

The most recent significant flooding in Denver occurred in May of 1973 when steady rains swelled channels and culverts resulting in $50 million in damages (Hansen, 1973). Scour in the South Platte River undermined and destroyed the 15th Street bridge (Table 5).

The devastating flood of June 1965 resulted in two significant achievements in drainage design and control. First, in 1968, the Denver Regional Council of Governments contracted for the preparation of an Urban Storm Drainage Criteria Manual (Wright-McLaughlin Engineers, 1969). Second, the flooding led to the passage of Colorado Senate Bill 202, the Urban Drainage and Flood Control Act of 1969, creating the Urban Drainage and Flood Control District. The District is authorized to set a 0.4 mill tax levy for floodway and floodplain engineering, maintenance, and master planning for Denver and surrounding metropolitan areas. Drainage and flood control are the only responsibilities of the District.

In 1935–1936 Kenwood Dam, or Sullivan Barrier, was constructed on Cherry Creek at a point now located just outside the southeast city limits of Denver. The dam cost about $800,000, of which Denver paid approximately 75 percent. However, during the construction of this dam, the infamous storm of May 30–31, 1935, occurred in the adjoining Republican River Basin. This storm far exceeded any other in historical times, and the Kenwood Dam was considered underdesigned and obsolete before it was completed (Costa, 1978). In 1950 the U.S. Army Corps of Engineers completed the present Cherry Creek Dam and Reservoir at a cost of $14.8 million (Figure 1; Table 7). The dam is an earthfill structure, 14,300 ft (4,359 m) long and 140 ft (42.7 m) high, dwarfing the pre-existing Kenwood Dam (Figure 18). In 1965 the dam completely impounded a flood of 59,000 cfs (1,671 cms) along Cherry Creek which would have caused an estimated $130 million in damages to Denver downstream. Unfortunately, encroachment and development along the channel downstream and along the spillway outfall has re-

Table 6. *Floods along South Platte River and Cherry Creek.*

Date		Stream	Peak Q cfs	Note
	1844	So. Platte River	?	Earliest historical flood
May	1864	Cherry Creek	20,000 ?	19 killed; all bridges across Cherry Creek destroyed
June	1864	So. Platte River	?	Heavy rain on snow in upper basin
May	1867	So. Platte River	?	Greater than 1864 flood
May	1876	Cherry Creek	11,000 ?	—
		So. Platte River	?	Great destruction
May	1878	Cherry Creek	?	Less than 1864 flood; all bridges across creek destroyed
May	1885	Cherry Creek	20,000	Largest historical flood
June	1894	So. Platte River	14,000	—
July	1912	Cherry Creek	11,000–15,000 ?	Over ½ million dollars damages in Denver
June	1921	So. Platte River	8,790	500 homes inundated in Denver
Aug	1933	Cherry Creek	16,000	Failure of Castlewood Dam; $800,000 damages in Denver
Sept	1933	So. Platte River	22,000	—
June	1965	So. Platte River	40,300	Six drowned; $300 million damages in Denver. Largest historical flood.
May	1973	So. Platte River	18,500	$50 million damages; 1.1 times 50-year flood

duced the flood control benefits of the dam (Costa, 1978).

After the 1965 flood, the $85 million Chatfield Dam was constructed on the South Platte River just south of Denver (Figure 1) for flood control and incidental recreation (Table 7). Construction was started in 1967 and was completed in 1977. In 1979 Bear Creek Dam on Bear Creek, just east of the mountain front (Figure 1), was finished. This completed the damming of most major streams draining through the Denver area (Table 7). Only Clear Creek remains undammed.

SEISMICITY

Colorado has long been considered an area of low seismicity, with only a minor potential for future damaging earthquakes (Algermissen, 1969). Recent investigations, however, have discovered several active faults that are capable of generating future earthquakes and numerous other faults that are suspected of being active (Kirkham and Rogers, 1981; Shaffer, 1980; Ostenaa et al., 1980). These investigations suggest Colorado is a moderately active earthquake area; and, in time, larger earthquakes than have yet been experienced can occur (Simon, 1969).

Modern man has occupied Colorado for about 120 years, and during this period hundreds of earthquakes have been noted. Over the past few decades, Colorado earthquakes have been detected, located, and measured by a small number of seismographic instruments. Most earthquakes have been minor, but some exceeded Richter magnitude 5, with locally severe ground shaking. Father Armand W. Forstall installed the first seismograph in Colorado at Regis College in Denver in 1909. This instrument has provided valuable data but has op-

Table 7. *Major flood-control dams in the Denver metro area.*

	Cherry Creek Dam	Chatfield Dam	Bear Creek Dam (Mt. Carbon)
Date completed	1950	1976	1979
D.A. controlled	386 mi²	3,018 mi²	262 mi²
Type	Earth fill	Earth fill	Earth fill
Height	140 ft	147 ft	179.5 ft
Length	14,300 ft	13,340 ft	5,300 ft
Vol. of fill	13,240,000 yd³	14,650,000 yd³	11,345,000
Spillway type	Uncontrolled side channel	Ungated concrete chute	Ungated dirt (bedrock) chute
Max. capacity	(185,000 a.f.*) 93,000 a.f.	355,000 a.f.	75,000 a.f.

* When originally built. Urban encroachment has reduced maximum capacity by rendering spillway unusable.

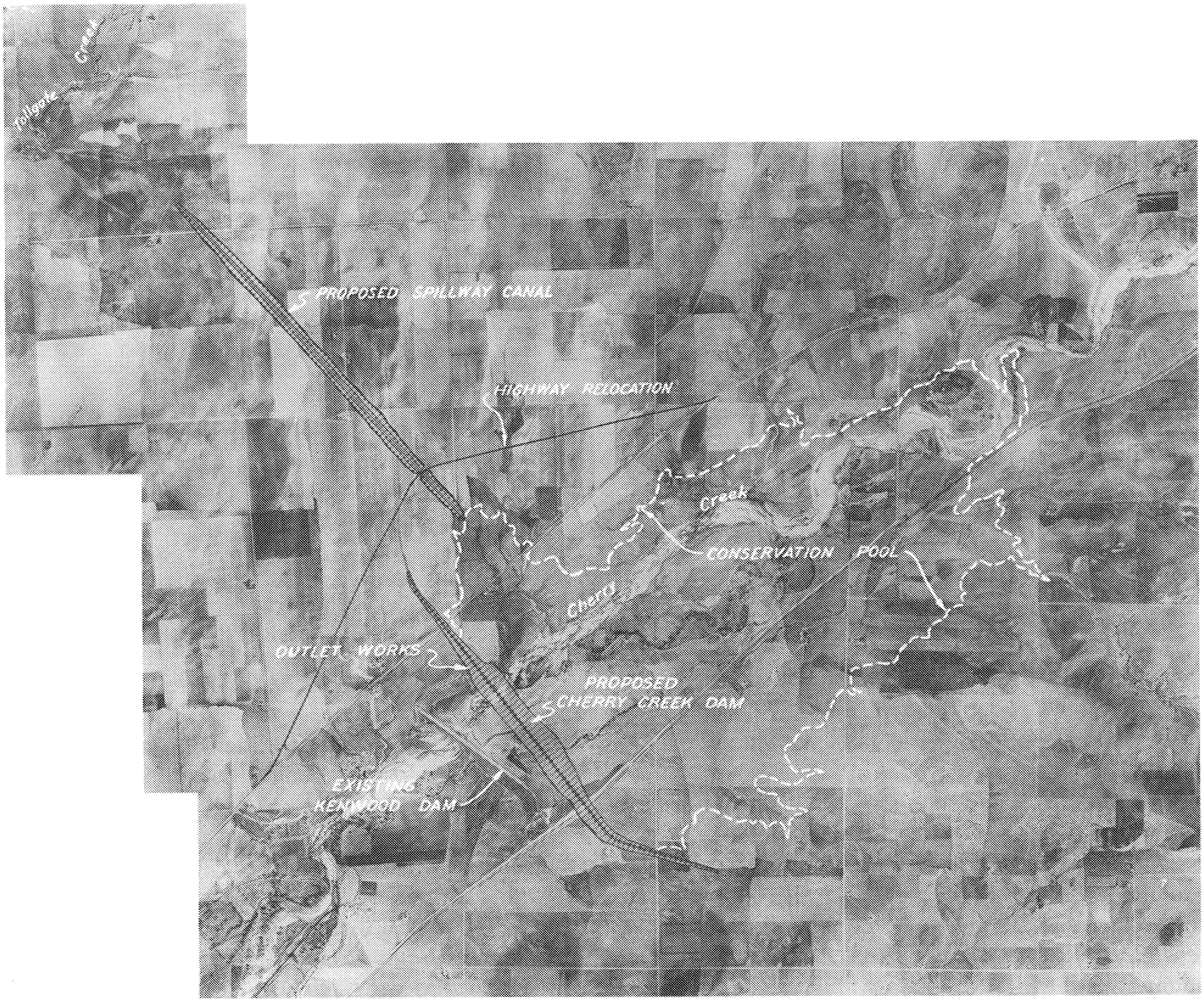


Figure 18. Vertical airphoto taken in the late 1940's showing the plan of the Cherry Creek Dam and Reservoir. Compare the size of the new dam with the then-existing Kenwood Dam. (Photo courtesy of U.S. Army Corps of Engineers.)

erated generally at low gain and is capable of detecting only large events. A seismograph was in operation at the University of Colorado at Boulder from 1954 to 1959. In December of 1961 the Colorado School of Mines installed a three-component seismograph at Bergen Park. This seismograph has operated at high gain since installation and is the primary source of instrumental data in Colorado. For a period during 1971 and 1972, the Colorado School of Mines and NOAA jointly operated a seven-station, state-wide network. One strong motion accelograph is currently operational in the Denver metropolitan area and is located in the Regency Inn (Figure 1).

Many earthquakes have been felt or instrumentally located in the Denver metropolitan area. Probably the largest of these events occurred on November 7, 1882 (Hadsell, 1968). It was felt throughout Colorado and in several adjacent states. Modified Mercalli Intensities of VII were reported in the Denver area. Most accounts of the earthquake suggest it was centered north of Denver, possibly near present-day Broomfield or Louisville. One recent evaluation suggests that the epicenter may have been in northwest Colorado, not in the Denver area (Dames and Moore, Inc., 1981). Although there were widespread, but scattered reports of violent ground shaking, relatively little property damage apparently resulted. This is probably due to the sparseness of development and prevailing earthquake-resistant one- or two-story frame construction of that time. A similar Intensity VII earthquake today could possibly result in millions of dollars of property damage, and perhaps loss of life.

In September of 1961, the U.S. Army drilled a 12,045-ft (3,671.3-m) injection well on the Rocky Mountain Arsenal property, and in March 1962 began to dispose of contaminated wastewater from its chemical manufacturing plant (Figure 1). Maximum injection pressures were 550 to 1,050 lbs/in.^{2} (379×10^4–724×10^4 N/m^2) with an injection rate of 200 to 300 gal/min (12.6–18.9 liters/s) (Evans, 1966). Beginning on April 24, 1962, and extending into 1968, Denver metropolitan area, which had not had a felt earthquake in 80 years, began to experience earthquakes at a rate of from 10 to over 100 per month. Most of the epicenters were within five mi (8 km) of the Arsenal well. Initially, the earthquakes were very small, and only a few were felt. The two largest earthquakes occurred on April 10, 1967 (Richter M = 5.0; I = VI), and August 9, 1967 (M = 5.3; I = VII). These quakes did considerable damage in the Commerce City and Northglenn suburbs north of Denver. In November 1965, D. M. Evans, a Denver-based consulting geologist, publicly expressed the view that the ongoing series of earthquakes were caused by the wastewater injection into the Rocky Mountain Arsenal well. His conclusion was based on the direct temporal correlation between the rate of fluid injection at the well and local earthquake frequency.

Because of increased awareness of the potential for damaging earthquakes and the possible relationships between the earthquakes and the disposal well, the U.S. Geological Survey, in cooperation with the Colorado School of Mines, was directed to evaluate the earthquake series. This study involved review of the pre-injection earthquake history of the area, and the establishment of a dense microearthquake detection network to accurately locate all events (Healy et al., 1966). No absolute evidence of any pre-injection seismic activity near the Rocky Mountain Arsenal was found, but two felt earthquakes were suspected of having occurred nearby. Sixty-two earthquakes were located by microearthquake monitoring during the Federal investigation that clustered in a seven-mi by two-mi (11 by 3 km) ellipsoidal zone that included the disposal well. This earthquake trend probably coincides with, and roughly defines, a zone of faulting or fracturing deep in the sub-surface (Kirkham and Rogers, 1981). The U.S. Geological Survey (Healy et al., 1966) concluded that there were definite temporal and spatial relationships between the disposal well and the series of earthquakes.

As a result of this postulated cause and effect relationship, fluid injection of the well was terminated on February 20, 1966. The earthquakes, however, continued to occur. The largest and most damaging earthquake, with a magnitude of 5.3, happened over a year after injection was halted. This apparent discrepancy was explained by Healy et al. (1968) using a conceptual fracturing model that suggests the larger earthquakes should occur after cessation of injection.

Most workers who have studied the Rocky Mountain Arsenal earthquakes believe that the fluid injection triggered the earthquakes (Healy et al., 1966, 1968; Hollister and Weimer, 1968). Considerable evidence has been introduced that indicates tectonic stresses existed in the area prior to injection and that the fluid injection triggered a partial release of this stored energy (Healy et al., 1968; Wyss and Molnar, 1972; Hsieh and Bredehoeft, 1979). The Denver earthquakes might have occurred even if the Arsenal well had not been drilled, and wastewater had not been pumped into the subsurface (Hollister and Weimer, 1968). This interpretation is supported by the recurrence of small-magnitude earthquakes in the northwest Denver suburbs of Thorton and Northglenn. On April 2, 1981 an earthquake of magnitude 4.1 occurred in this area and caused some minor damage. Thus, the possibility of future tectonic stress accumulation and release in the northeast Denver area cannot be ruled out at this time. The maximum magnitude of future earthquakes would probably be at least equal to the previous events (magnitudes 5.0 to 5.3) but could possibly be larger.

The entire State of Colorado is classified as "minor damage" in the seismic risk map of the United States (Algermissen, 1969). This classification implies the following seismic risk: "minor damage; distant earthquakes may cause damage to structures with fundamental periods greater than 1.0 second; corresponding to intensities V and VI on the Modified Mercalli Intensity Scale." Present building codes in Denver follow the Unified Building Code which adopts the risk map of Algermissen (1969) for seismic resistant design. Denver may, therefore, be facing a serious problem in the event of a moderate or major earthquake. Matthews (1973) argues this seismic rick classification is too

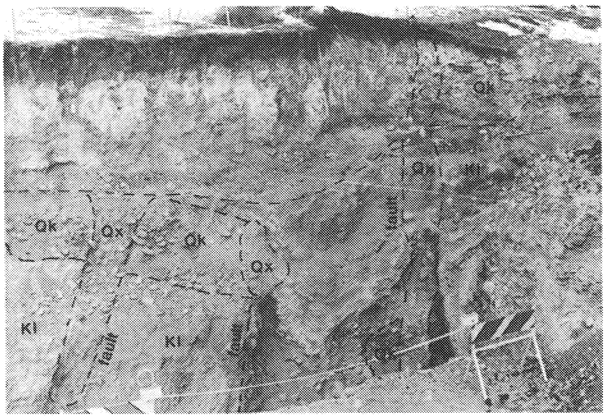


Figure 19. Photograph of east side of trench excavated across Golden Fault near Golden (Figure 1). Note faulted Kansan (?) deposits (Qk), especially the 18 ft (5.5 m) of offset along the far right fault. Zones of disoriented gravel clasts are labeled (Qx), and the Cretaceous Laramie Formation, (Kl) (generalized from Kirkham, 1977).

low for Denver because: (a) based on recent investigations of Cenozoic geology, the Rocky Mountains of Colorado should today be considered an active tectonic area; and (b) the 100-yr seismic history record in Colorado is too short to rule out a major or moderate earthquake in or near populated areas. Indeed, in 1967 (Intensity = VII) and perhaps 1882 (?) (Intensity = VII), earthquakes occurred in Denver with resulting intensities that exceeded the seismic design parameters specified in the city building code.

Alternative evaluations of seismic risk for Denver, based primarily on historic records, are: (a) Modified Mercalli Intensity VII or greater, and Richter magnitude 5.0 or greater, with a frequency of roughly four per 10 yrs, per square degree of surface area (Simon, 1972); and (b) horizontal acceleration of 0.04 g in rock with 10 percent probability of being exceeded in 50 yrs (Algermissen and Perkins, 1976).

Concern for the seismic safety of the Denver metropolitan area has resulted in several detailed studies of the Golden Fault west of Denver (Figures 3A and 3B). Scott (1970, p. C18) concluded that the Golden Fault could produce earthquakes having intensities greater than V. Recent excavations along the Golden Fault by the Colorado Geological Survey (Kirkham, 1977) indicated at least two periods of fault rupture with a total of 18 ft (5.5 m) of vertical displacement since the Yarmouth (?) interglacial. The most recent movement along the fault post-dates a layer of 600,000-yr old volcanic ash and overlying colluvium, but is believed to pre-date the surface soil of Sangamon (?) age (Kirkham, 1977) (Figures 1 and 19).

A major study recently completed concludes that the Golden Fault is not a capable fault and could not cause an earthquake strong enough to damage the Rocky Flats nuclear processing plant located along the fault trace northwest of Denver (Dames and Moore, Inc., 1981). Microearthquake monitoring of Chatfield and Bear Creek Reservoirs southwest of Denver has been conducted for the Corps of Engineers. No definite local earthquakes have been recorded (Patrick, 1977).

Earthquake insurance is generally available in Denver at rates of about \$0.44 per \$1,000 for frame structures, and \$0.68 per \$1,000 for all other buildings, with a 5 percent deductable.

ENVIRONMENTAL CONCERNS

Water Supply

The first water supplies for the City of Denver came from springs, shallow wells, and directly from Cherry Creek and the South Platte River. Addison Baker homesteaded 160 acres in 1866 around a large spring above the mouth of Cherry Creek. The spring had a daily output of 100,000 gal (378,500 l) and Baker delivered some of the water to residents of Auraria. This was Denver's first commercial water supply (Smiley, 1901).

In 1872 the Denver City Water Company piped water directly to houses from a large shallow well in Cherry Creek. Water was delivered through four mi (6.4 km) of wooden mains by a steam-driven pump. The rapid growth of population in the early history of Denver meant increasing demands on the water supply system; and for the next two decades, 11 private water companies competed to supply water from local rivers, ditches, and wells. One water company even provided free water to its customers for two years between 1889 and 1890 in an attempt to drive competitors out of business.

Some supply schemes included gates and ditches on the South Platte River about three mi (5 km) south of Denver to divert water to a lake from which it was pumped into mains beneath city streets. In 1887, infiltration galleries were constructed in the bed of Cherry Creek east of Denver. However, shallow wells and surface water supplies gradually became polluted.

The problem was temporarily solved in 1883. In March of 1883, R. R. McCormick was boring for coal near St. Luke's Hospital in north Denver. He

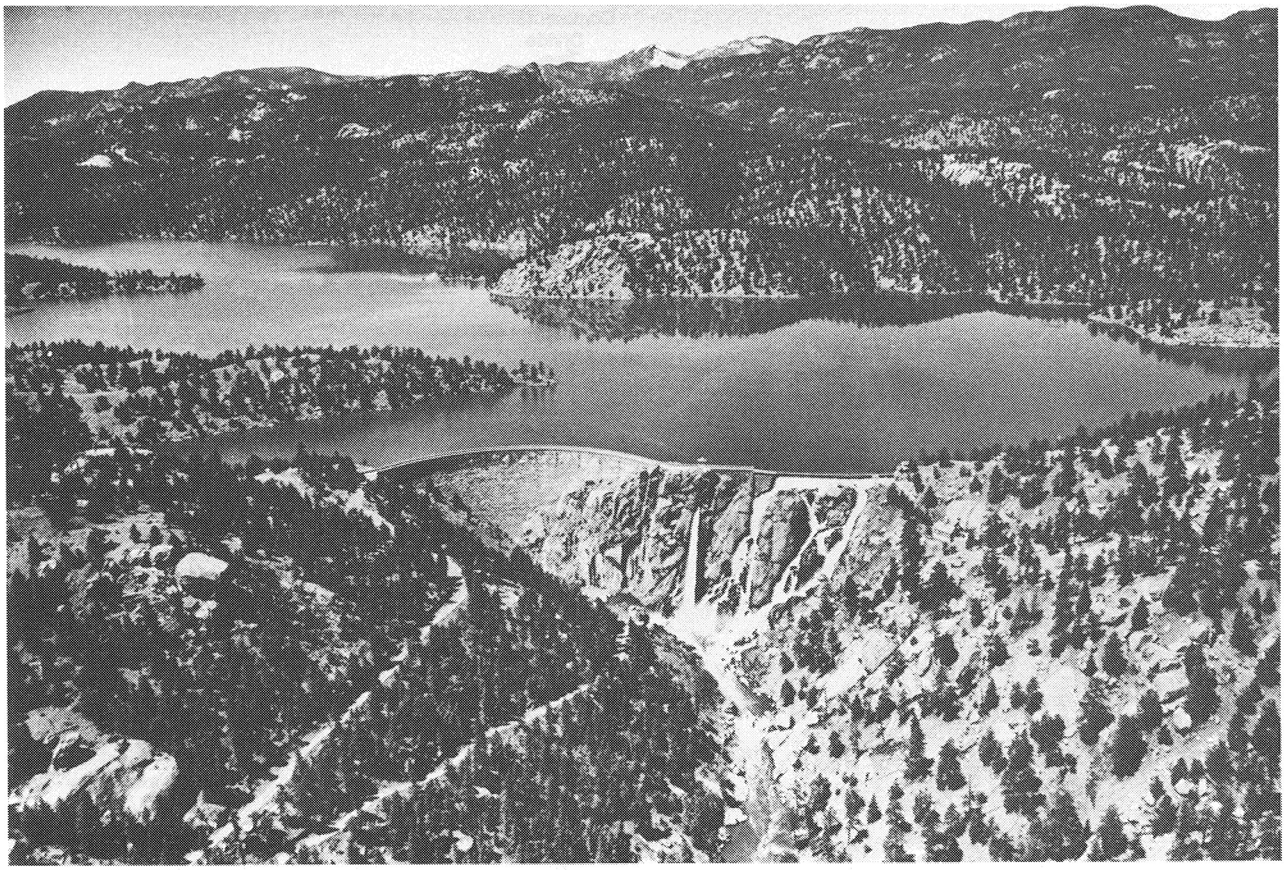


Figure 20. Cheesman Dam and Reservoir, a National Historic Civil Engineering Landmark. (Photo courtesy of the Denver Water Board.)

was forced to abandon the hole because a large flow of artesian water prevented further drilling. The ground water came from the Denver/Arapahoe aquifer underlying the Denver Basin at depths ranging up to 1,500 ft (457 m) and was of markedly purer quality than that delivered by the Denver City Water Company from the South Platte River (Cross et al., 1884). Other deep wells were soon drilled by brewers such as Zang and Tivoli, department stores such as Daniels and Fisher, and hotels such as the Brown Palace, as well as by private individuals. By 1900, more than 400 wells had tapped water-bearing zones in the buried aquifer at depths of 375 and 600 ft (114–183 m). Drilling costs were about $2 per ft (0.3 m) of depth (Cross et al., 1884).

Since the beginning of extensive use of the Denver/Arapahoe aquifer in 1883, the artesian head has declined under the city by approximately 400 ft (122 m) (U.S. Geological Survey, 1968). Extensive use is still made of ground water in Denver. The Brown Palace Hotel recently drilled a new well when its old one became sand-plugged. Ground water is still used by a private water company who delivers bottled artesian water. Water from wells is used in the laundry of a large hospital in Denver. The ground water is so soft it can be used successfully in boilers as well as kidney-treatment dialysis machines. Recently the hospital laundry had to double its soap consumption when using harder city water because the hospital well was shut down temporarily.

In 1894, the remaining water companies serving Denver merged to form the Denver Union Water Company, managed by W. S. Cheesman. The major contribution to Denver's water system by this private water company was the construction of Cheesman Dam and Reservoir on the South Platte River about 48 mi (77 km) southwest of Denver (Figure 20). Construction began in 1900 and was completed in 1905. The dam rises 222 ft (68 m) above the stream bed and is 1,100 ft (335 m) long including

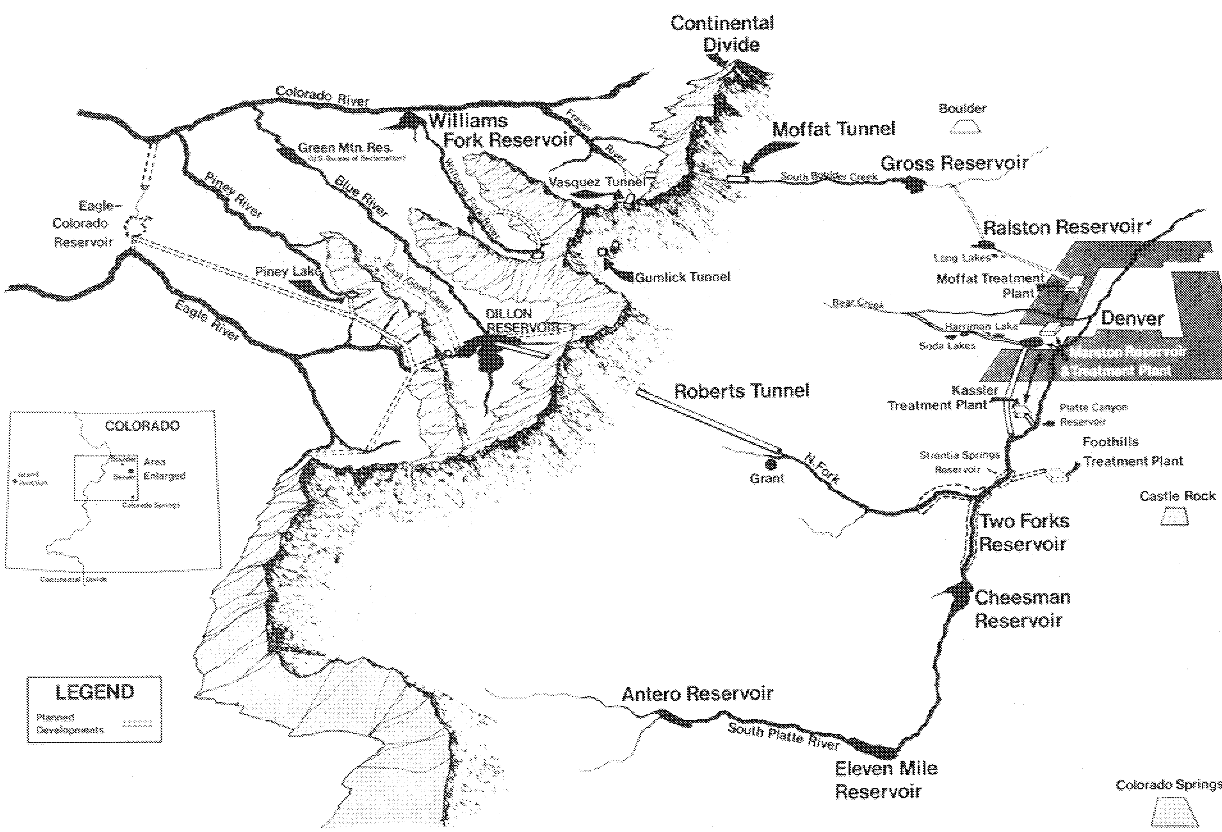


Figure 21. General plan of the Denver water supply system. (Diagram courtesy of the Denver Water Board.)

the spillway. The dam is constructed of locally-quarried granite blocks laid in cement mortar. Outlet works were tunneled into the rock abutting the dam, and water topping the spillway cascades over granite cliffs to the stream bed below. When completed in 1905, Cheesman Dam was the highest gravity-arch stone masonry dam in the world and provided the first substantial and continual on-stream storage of raw water for municipal use in the Rocky Mountain west. Cheesman Dam was designated a National Historic Civil Engineering Landmark by the American Society of Civil Engineers (ASCE) in 1973.

Denver's first significant water treatment facility consisted of underground infiltration galleries constructed in 1890 at Kassler, south of Chatfield Dam. The facility was rebuilt in 1906 and became the first plant west of the Mississippi River to employ the English slow-sand filter process. In 1979, it was designated a landmark by the American Water Works Association.

In 1918, Denver citizens voted to issue bonds for the purchase of the Denver Union Water Company. They also approved a management plan placing control of the system under an independent, nonpolitical five-member Board of Water Commissioners appointed by the mayor for staggered six-year terms. The private water company was purchased for $14 million, which, in 1918, operated 613 mi (986 km) of conduits and water mains, a small pumping station and filter plant, and one storage reservoir on the South Platte River (Denver Water Department, 1976).

The Denver Water Board began buying water rights and acquired Antero Reservoir on the South Platte River in 1924 for $450,000. The following year, the Marston Treatment Plant was completed, and in 1932, Eleven Mile Canyon Dam, the third dam on the South Platte River, was built. This completed the initial development of the South Platte River as a water supply for Denver (Figure 21).

The Denver Water Board was a far-sighted organization. In 1922, the Colorado General Assembly passed the Moffat Tunnel Improvement District Bill to help construct a railroad tunnel through the Continental Divide and connect Denver with Salt

Table 8. *Major storage reservoirs and treatment plants, Denver water supply.*

	Capacity	% of Total	Completed	Dam Type
Reservoir				
Antero	15,878 a.f.	3.0	1909	Earth-fill
Cheesman	79,064	15.2	1905	Gravity arch masonry; with granite facing
Dillon	254,036	48.7	1963	Earth-fill
Eleven Mile	97,779	18.8	1932	Gravity arch concrete
Gross	41,811	8.3	1954	Gravity arch concrete
Marston	17,213	3.3	1902	Earth-fill
Ralston	11,272	2.1	1937	Earth-fill
(Strontia Springs)	(7,700)	—	(1982)	Thin arch concrete
Treatment Plant				
Kassler	50 mgd	9.6	1890; 1906	Slow sand filter
Marston	260 mgd	50.0	1925; 1961; 1967	Rapid sand filter, with micro strainers
Moffat	210 mgd	40.4	1937	Rapid sand filter, with micro strainers
(Foothills)	(125 mgd)	(19.4)	(1982)	Rapid sand filter

Lake City. The City of Denver got two features incorporated into the design of the tunnel: (a) it was to be constructed at 9,000 ft (2,743 m) rather than at a much higher altitude as favored by many transportation experts, and (b) the pioneer bore method of advance would be used to act as a service tunnel with cross-cuts drilled to the main bore. In 1928, when the railroad tunnel was completed, the Denver Water Board leased the pioneer bore, enlarged and lined it for the purpose of transmitting water from the Fraser River system on the western slope through the tunnel under the Continental Divide and into Denver (Figure 21). In June 1936, the first water flowed through the Moffat Tunnel to help alleviate the drought of the 1930's. In 1937, the Moffat Treatment Plant was completed to process this new water supply, and two reservoirs were constructed in 1937 and 1955 to provide storage (Figure 21). In 1979, the Moffat Tunnel was declared a National Historic Civil Engineering landmark by the American Society of Civil Engineers.

In 1946, another major transmountain diversion was begun with initial construction of a 23.3-mi-(37.5-km) long tunnel to transport water from the Blue River system on the western slope under the Continental Divide into the South Platte River. In 1956, six large construction companies won a joint-venture contract to finish the tunnel which they completed six years later. The Harold D. Roberts Tunnel, named after a Denver lawyer who secured many water rights for the city, was, when constructed, the longest underground water tunnel in the world (Wahlstrom, 1981). Dillon Dam and Reservoir were constructed between 1959 and 1963 to store western slope water for transport through the Roberts Tunnel. This reservoir doubled Denver's water storage (Table 8).

Colorado follows the Appropriation Doctrine of water rights; the purchase and transfer of these water rights, principally from agricultural to municipal use, is the way most domestic water supplies in Colorado have evolved (Cox, 1967). The City of Denver supplies water to several surrounding communities, and it was this ability to receive water that became the major incentive for annexation. But in 1951, prompted by impending water shortages, Denver defined an area surrounding the city outside of which water service would not be extended. This became known as the infamous "blue line," which forced the development of small, independent, and sometimes marginal, new water systems in the metropolitan area (Cox, 1967).

Total raw-water storage capacity of the Denver system in 1979 was 530,943 a-ft (655 hm), whereas total reservoir storage over the past five years has ranged from 58 to 89 percent of capacity (Denver Water Department, 1979). This captured water must be treated before distribution to customers, and therein lies the major problem of the Denver water supply.

During a two-week period in the summer of 1973, the city water treatment facilities were overtaxed on five days. In 1977, a mandatory water conservation program was begun because of inadequate treatment capacity and below-normal spring runoff. Since an estimated 40 percent of residential water is used for lawn watering, such watering is allowed only every third day during the summer. Water consumption in 1979 averaged 197 gallons per capita per day (gcd) (746 lcd), compared to an all-time high of 225 gcd (852 lcd) in 1974. The Denver Water Board also began a five-year tap allocation system whereby the number of new three-quarter inch water taps installed between 1977 and 1981 would not exceed 26,000. Denver Water Department customers comprise 40 percent of the state's population, yet account for only 1.3 percent of Colorado's water use. Agriculture is by far the largest consumer in the region.

A fourth water-treatment facility, the Foothills Complex, has been delayed five years because of environmental considerations. Construction finally began in 1979 and is scheduled for completion in 1982. The Foothills Complex includes a diversion dam in the South Platte Canyon at Strontia Springs, a connecting 3.4-mi (6.4 km) long tunnel to an initial 125 million gallons per day (mgd) treatment plant, and a conduit to bring the treated water to Denver. Ultimate treatment capacity is to be 500 mgd. The Foothills Complex will deliver water to the Denver metropolitan area by gravity flow and will also generate hydroelectric power (Figure 21).

In 1981, construction was also begun on a 1 mgd demonstration plant to recycle sewage effluent into potable water. Initially, none of the recycled water will be put into the city's water supply, although it will be made available for recreational and industrial uses.

The Denver water system is self-sustaining financially. No sales, property, or other tax dollars go into its operation. In 1980, water rates increased 29.3 percent for metered and flat-rate customers in Denver, 52 percent for residential users outside the city, and 50 percent for tap fees. About 37 percent of Denver City customers are metered. The remainder pay flat rates based on size of house, number of rooms, and number and kind of water-use devices. Annual water bills in Denver average $184.

Wastewater Disposal

The first sanitary sewer was constructed in downtown Denver in 1891. This same sewer is still the primary main to the Denver wastewater treatment plant downtown along the South Platte River. Between 1891 and 1936, wastewater from Denver was discharged, largely untreated, into rivers and streams. In 1936, the Denver Northside Wastewater Treatment Plant was completed as a primary treatment plant capable to treating 50 mgd (2.2 cms) (Figure 1). In subsequent years, the plant was expanded and modified and today can handle an average capacity flow of 106 mgd (4.6 cms) and a peak capacity flow of 160 mgd (7 cms). Average flows, however, have averaged 90 to 100 mgd (3.9 to 4.4 cms) and peak flows 135 to 140 mgd (5.9 to 6.1 cms). The Denver Northside Plant now handles 90 percent of Denver's wastewater flow, serving approximately 650,000 people. In 1979, the last remaining combined storm sewer/sanitary sewers were eliminated, giving Denver a completely separate wastewater system.

Primary treatment at the Denver Northside Plant consists of five mechanical processes. Five mechanically-cleaned, one-in. (25 mm) screens remove large solids arriving from the interceptors such as cans, papers, and debris. Heavy inorganic solids (grit) such as sand are removed, washed, and disposed of in landfills. The sewage is then pre-aerated by bubbling forced air through the liquid to reduce odors, bring grease to the surface, and help aggregate fine suspended solids. The liquid waste from the pre-aeration tanks is then transferred to settling basins where most of the suspended solids are removed. Each tank has a scraper to remove coarse solids from the tank bottom and a skimmer to remove floating grease from the surface. The liquid effluent is then transported by gravity flow in pipes to another plant for secondary treatment, and the solid sludge is pumped to digesters. There, anaerobic organisms decompose organics to more stable materials, producing methane gas which is used as an energy supply for the plant. Future plans at the Northside plant are to use sludge gas to operate 1,000-kw dual fuel, engine-driven generators that will eventually produce enough electricity to meet the entire plant's energy requirements. The primary treatment removes about 60 to 65 percent of the solids and about 30 percent of the incoming wastewater's biochemical oxygen demand (BOD).

Digested sludge is then pumped to another plant downstream for secondary treatment where it is processed and dried for ultimate disposal at the Lowry landfill east of the city (Figure 1). The long-range plan for sludge disposal is a land treatment facility in adjoining Adams County. However, this plan still requires permits and approval.

In 1966, the Metropolitan Denver Sewage Disposal District (MDSDD), consisting of 21 municipalities, of which Denver is the largest, completed the MDSDD No. 1 plant, a secondary wastewater treatment plant serving 1.1 million people in the metropolitan area (Figure 1). With the completion of this facility, Denver ceased discharging its effluent into the South Platte River and constructed an effluent pipeline to the MDSDD plant for secondary treatment.

About 10 percent of Denver's wastewater flows directly to the MDSDD plant, and the Denver Northside Treatment Plant has an overflow system which takes excess flows directly to the MDSDD No. 1 plant.

The secondary treatment consists of activated sludge with aeration using compressed air and pure oxygen and settling tanks. Incoming wastewater BOD averages 200 to 300 ppm, while discharged wastewater from the plant averages 20 ppm, representing 90 to 95 percent BOD removal. The plant handles 150 mgd (6.6 cms) average and has a capacity of 170 mgd (7.4 cms).

Processed, dried sludge from the plant averages 100 dry tons (91 tonnes) per day. Most all of this is hauled by truck to Denver's Lowry landfill 15 mi (24 km) east of the city for landfarming disposal (Figures 1 and 22), but small amounts go to the Colorado State University experiment station near Greeley, Colorado, for agricultural research, and to the Denver Parks system for fertilizer and soil conditioner. Sludge from this plant also was used by a mining company for reclamation of tailings from molybdenum mining in the mountains west of Denver.

Wastewater and sewage rates in Denver are based on water usage during the winter billing period (November to February) at $0.95 per 1,000 gal (3,758 l) of water usage, or $5.19 minimum, whichever is greater. For homes without water meters, flat rates are assessed based upon house size, number of rooms, and number of water-use devices.

Solid Waste Disposal

In 1980, the City and County of Denver operated no sanitary landfills, and no solid wastes were disposed within the city limits of Denver. The city collects household rubbish and other waste materials, hauls them to landfills surrounding the city, and pays tipping fees. This has not always been the case in the past. Much high-value real estate in the City of Denver has been developed over old landfills within the city limits, including major shopping centers, municipal facilities, warehouses, light industries, sports arenas, parks, and residential structures (Figure 14). These former landfills are concentrated along the valleys of the South Platte River and Cherry Creek (McBroome and Hansen, 1978). The City and County of Denver still owns a landfill on the Lowry Bombing Range east of the city in adjacent Arapahoe County, but the landfill is operated by a private contractor.

Table 9. *Solid waste disposal in Denver, 1979.*

Location of Landfill (Figure 1)	Amount (Tons) of Waste	Percent of Total
Property Investment Sanitary landfill, Adams Co.	77,077	40
Arapahoe Co. Sanitary landfill	77,077	40
Rooney Road Sanitary landfill, Jefferson Co.	19,269	10
Lowry Bombing Range landfill, Arapahoe Co.	19,269	10

In 1979, Denver collected 192,693 tons (174,811 tonnes) of household rubbish from 142,000 homes. Rubbish collected averages 400 tons (363 tonnes) per day in winter and 800 to 900 tons (726–816 tonnes) per day in summer. Assuming 2.4 people per home, that means 340,800 people were served; and residents generated an average 3.1 lbs (1.4 kg) per person per day. The City of Denver spent about $9 million on waste disposal in 1979, which amounts to about $26.40 per person served per year.

Denver utilized four sanitary landfills for disposal of household rubbish in 1979 (Table 9) (Figure 1). Three of these landfills are nearly full now, so in the next five years Denver will face a major shift in the location of its solid waste disposal to the enormous Lowry landfill. This site was formerly a practice bombing range for pilots in training at Lowry Air Force Base during World War II. Denver bought the land from the Federal government after the war. This landfill is 2,800 a (11.3 km^2) and has a projected life of 100 years. However, shifting waste disposal exclusively to this landfill would mean an additional 100,000 mi (160,900 km) per yr hauling for Denver.

Feasibility studies have been completed for the design of an incineration plant closer to the city capable of burning 300 tons (272 tonnes) per day of solid waste. The plant would generate steam to be sold to a utility company, to be used in generation of electricity.

In the Denver metropolitan area, 80 percent of the solid wastes now being generated is estimated to consist of organic materials (Ralph M. Parsons Company, 1976, pp. 2–26). The hazards of methane gas produced in landfills has been discussed earlier, but in Adams County, which adjoins Denver on the north, gas is being considered a resource rather than a hazard. The county received a U.S. Department of Energy grant to investigate methane recovery feasibility in seven old landfills. The studies have shown that five of the seven landfills are potentially good sites for recovery and profitable use of methane gas (SCS Engineers, Inc., 1980). At two sites, gas is actively being ventilated to the atmosphere today. These five landfills contain an estimated 9.85 million tons of solid waste, which is capable of producing 0.15 ft^3 (0.004 m^3) of gas per pound per yr which is 45 to 50 percent methane. A total volume of three ft^3 (0.08 m^3) of gas can be recovered from each lb (0.45 kg) of waste, which indicates gas could produce $1{,}400 \times 10^6$ ft^3 (40×10^6 m^3) of methane per yr, containing 450 to 500 BTU per ft^3 (compared to 850 to 1,000 BTU per ft^3 for natural gas). This is enough energy to heat about 3,500 homes in Denver for a year. Colorado House Bill No. 1214 was subsequently passed by the State Legislature in April 1980, giving counties and municipalities power to explore, develop, produce, distribute, market, and finance landfill-generated methane gas.

Hazardous Wastes

There are approximately 333,000 tons (302,098 tonnes) of hazardous wastes produced in the Denver metropolitan area each year (Hynes and Sutton, 1980). This is about 40 percent of the state's total hazardous-waste production. These wastes include acidic and brine solutions, heavy metal and oil sludges, contaminated wastewater, and solvents.

Since 1942, the Army Chemical Corps has operated a chemical manufacturing plant at the Rocky Mountain Arsenal just north of Denver (Figure 1). Between 1942 and 1957, contaminated wastewater from the Arsenal was contained in shallow evaporation ponds constructed in the permeable eolian surficial deposits underlying the Arsenal property. No attempt was made to seal these ponds. The result was severe pollution of the local shallow ground water. In 1957, the lagoons were sealed with asphalt, but this was not completely successful. Maximum migration rate of the wastewater was approximately three ft (1 m) per day. By 1960, an area of six and one-half mi^2 (16.8 km^2) extending to the northwest from the Arsenal to the South Platte River had been contaminated by chlorates and 2,4-D type compounds, both of which are effective herbicides. This resulted in extensive crop damage (Walker, 1961; Lindvall, 1979a, 1980).

Since December 1980, the Lowry landfill, owned by the City and County of Denver but operated by a private contractor, is the only approved site for disposing of non-radioactive hazardous wastes. More than 200 firms from the Denver area dump such wastes there.

The site is underlain by eolian sand, loess, alluvium, and Denver/Dawson mudstones and sandstones. Sewage sludge is either spread on the ground and plowed into the soil, or buried in bulk (Figure 22). Land disposal of sludge began in 1969, and by 1976, application rates ranged from 60 to 210 dry tons (54–191 tonnes) per acre (Robson, 1977). Liquid wastes were discharged into unlined earth trenches until several million gallons of liquid accumulated. The trenches were then filled with refuse and covered with a layer of earth (Robson, 1977).

By 1976, shallow stock-watering wells around the landfill were found to have markedly degraded water quality (Robson, 1977). The regional Fox Hills aquifer lies at a depth of about 1,800 ft (600 m) below the site and will not be affected by the disposal. The Colorado Geological Survey has subsequently classified the site as only marginally suitable for disposal for non-nuclear hazardous industrial wastes (Hynes and Sutton, 1980, plate I).

The Shell Chemical Company, a leased tenant of portions of the Rocky Mountain Arsenal, recently spent \$1.6 million for the construction of three clay-soil-lined evaporation ponds, covering 22 a (8.9 ha) adjacent to the Lowry landfill (Camp Dresser and McKee, Inc., 1978). The ponds were completed in 1980. An additional hazardous waste facility, incorporating storage cells for drummed waste, was under construction by a private operator in 1981.

The disposal of hazardous wastes in the Denver metropolitan area is presently in a state of turmoil. In 1968, Denver obtained its landfill designation certificate from Arapahoe County when there were no legal distinctions between solid and liquid, or hazardous and special wastes. In early December 1980, the Arapahoe County Commissioners passed a resolution giving Denver 10 days to stop the dumping of hazardous chemical and toxic wastes at the Lowry landfill. From then until 1980, there was a rising tide of public objection from residents in the vicinity. A court injuction now has temporarily halted the resolution of the County Commissioners. This

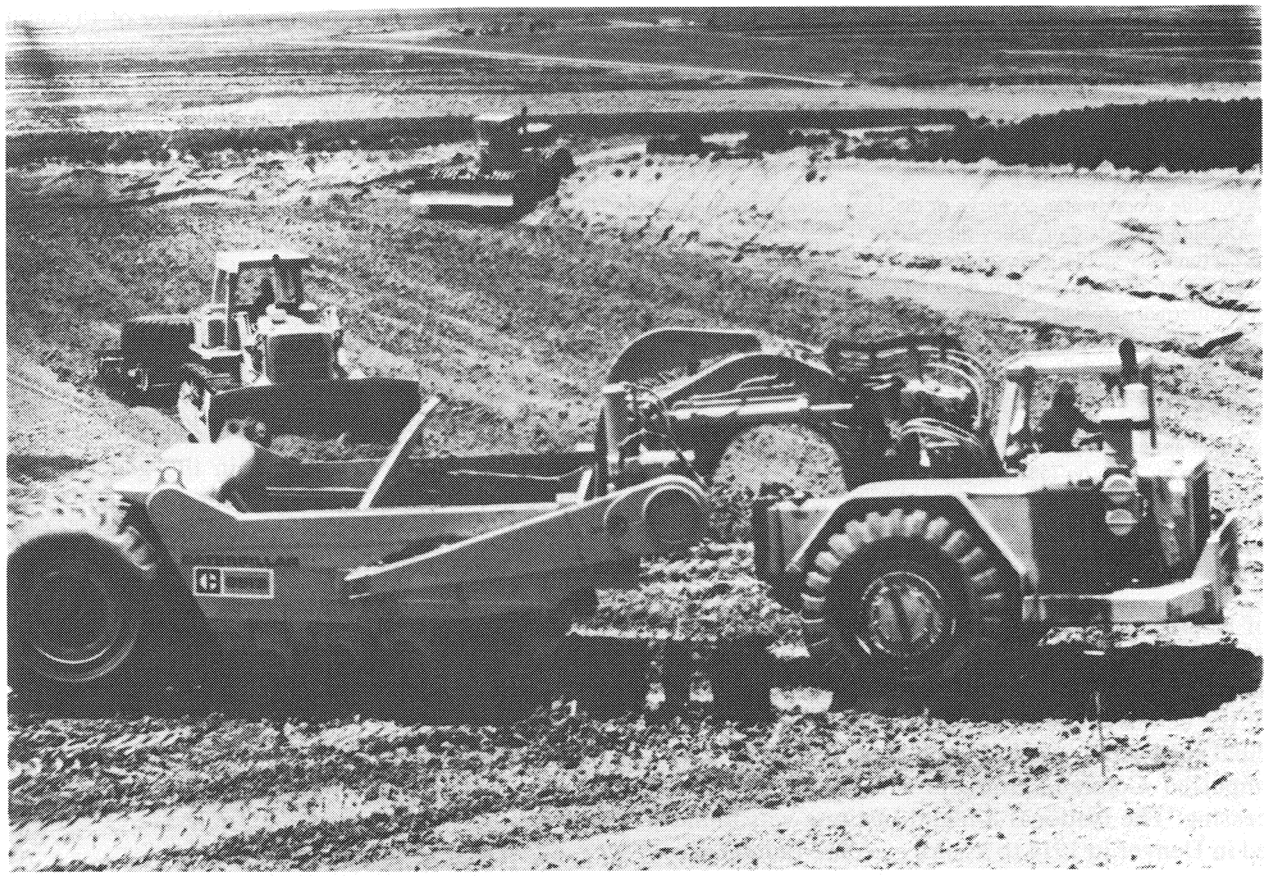


Figure 22. Construction underway at the 22 a (8.9 h), industrial brine evaporation pond facility of the City-County of Denver. The scraper pan in the foreground is transporting selected "suitable" materials from grading of an adjacent pond, to placement in a 6 in. (15 cm) lift for compaction-placement as bottom-liner fill. The facility is constructed entirely of silt and clay-size materials from the undifferentiated Denver/Dawson Formations, as encountered in site grading. Pond layout took into consideration stratigraphic and facies variations encountered in exploration and verified during construction. Laboratory determinations of the coefficient of hydraulic conductivity (permeability) of the engineered bottom-liner and key-trench cutoff fill indicated achievement of values less than 10^{-9} cm/sec at 95 percent of maximum dry density. The fill is being compacted by the sheepsfoot roller and scarified between lifts by the dozer, as shown in this view, taken in August of 1979 (Photograph by Allen W. Hatheway).

is a difficult situation, because there is no immediate alternative disposal facility in the entire state. North of Denver in Adams County, a site has been approved by the Environmental Protection Agency and the Colorado Department of Health, for hazardous waste disposal, but the site does not as yet have county approval and may not receive it.

The resolution passed by the Arapahoe County Commissioners also ordered the Metropolitan Denver Sewage Disposal District to halt dumping and spreading of sludge from its South Platte River plant by December 31, 1982. Local officials fear that the closing of the Lowry landfill to the dumping of hazardous wastes, without a nearby alternative, could precipitate a rash of "midnight dumping" of hazardous materials. Task forces presently are meeting to consider establishing a new operation with optimal geologic conditions within a reasonable distance of the Denver metropolitan area. Criteria set by the Colorado Geological Survey are shown in Table 10. Optimal conditions for potential storage of hazardous wastes near Denver are found in the upper and lower members of the Pierre Shale (Hynes and Sutton, 1980).

Radioactive Spoils

In February, 1979, the Environmental Protection Agency notified the Colorado Department of Health

Table 10. *Siting considerations for hazardous waste disposal in Colorado (from Hynes and Sutton, 1980).*

—Absolute containment for at least 1,000 years
—Minimum vertical thickness of 150 ft (46 m) of shale or clay with in-place permeability no greater than 0.1 ft/yr (30 mm/yr)
—Outside groundwater recharge or discharge areas
—Outside floodplain or valley-fill areas
—Tectonically stable; structurally simple, geologically
—Minimum of 1 mi (1.6 km) from any major fault, igneous, or geothermal activity
—Stable surface, not subject to erosion greater than 0.5 acre-feet per square mile per year (238 cubic meters per square kilometer per year
—Natural slopes between 2 and 5 percent
—Mean annual evaporation should exceed mean annual precipitation by 20 in./yr (508 mm/yr)
—Maximum 24-hour rainstorm should be no greater than 6 in. (152 mm)

of references found in three early U.S. Bureau of Mines Bulletins to a former "National Radium Institute" in Denver (Parsons et al., 1915). At the turn of the century, Germany and France were the principal suppliers of radium, and these countries even imported Colorado uranium ore for radium processing. The National Radium Institute was founded in Denver in 1914 to assure a secure radium supply for the United States, as World War I had begun. Radium had an alleged medical value for cancer treatments.

Over 10,000 tons (9,072 tonnes) of high-grade uranium ore (2.5 percent) were milled at the National Radium Institute's facilities between 1914 and 1917. In radium refining, uranium oxide was considered a waste product which, subsequently, was disposed of as artificial fill and construction material in the Denver area. Since the isotopes involved have half-lives of thousands of years, 60 years of dormancy has not greatly reduced the natural radioactivity.

Investigations in old telephone directories revealed that the former location of the National Radium Institute is the present site of a major privately-owned brick and tile storage and distribution facility in south Denver (Figure 1). With the cooperation of the owner, the State Department of Health visited the site and measured soil radioactivity of 1,600 microRoentgens per hour (Colorado Department of Health, 1981).

Further investigations identified 32 contaminated sites in Denver with maximum gamma intensities as high as 15,000 counts per second compared to natural background radiation in Denver of 15 counts per second. These sites include a restaurant parking lot, vacant lots, industrial and commercial property, and street subbases. The State Health Department has conducted drilling and soil sampling at nine sites, and estimates that a total of 35,000 yds^3 (26,775 m^3) of contaminated soil exists at these locations (Colorado Department of Health, 1981). The Denver radium-bearing contamination sites have been included in the recently passed (1980) federal superfund legislation to cover the costs of hazardous-waste clean-up.

Wetlands and Shore Protection

No wetlands or shorelines in the generally accepted geotechnical sense are present in the Denver area. Most low or poorly drained areas have been artificially dammed to raise the level of the many small reservoirs that exist in the area. Shore erosion is not a problem for such ponds, nor along the margins of the large flood-control reservoirs near Denver.

Major Engineering Structures

Some of the major engineering structures in the Denver area are tabulated in Table 5.

USE OF UNDERGROUND SPACE

There is only one significant use of underground space in the Denver metropolitan area, the storage of natural gas in old underground coal mines by the Public Service Company of Colorado.

In the early 1950's, the Public Service Company of Colorado, serving the Denver area, was faced with the need to store natural gas for peak-usage during the winter. After studying 16 storage sites, the utility company selected the old Leyden No. 3 coal mine located about 12 mi (19 km) northwest of Denver (Figure 1). Sub-bituminous coal occurs in the lower 200 ft (61 m) of the Laramie Formation at a depth of 700 to 1,000 ft (213–305 m) below the ground surface. Mining began in 1903, and coal mined from the Leyden mines was used primarily by the Denver Tramway Company for electrical power generation for trolley cars in Denver. Trolley service connected the mine to the city where coal was hauled in coal cars to electric-generating plants. The Leyden mine closed in 1950 after six million tons (5.4 million tonnes) of coal were mined, resulting in a void space of 150 million ft^3 (4.3 million m^3) (Meddles, 1978; Brown, 1978).

The Leyden mine was selected for underground storage of natural gas for several reasons. The mine was close to the Denver market, had adequate storage capacity, large volumes of gas could be withdrawn from a few wells, could be recharged quickly during off-peak demand times, and was located in a sparsely inhabited farming and ranching area (Figure 1).

After extensive testing indicated that the enclosing shale provided an impervious storage area, development of the mine as a gas storage facility began in 1959. By late 1961, natural gas was being stored successfully in the mine and continues to be so stored 20 years later. Storage capacity is presently 1.6 billion ft^3 (BCF) of gas with a maximum withdrawal of 185 MCF per day (Meddles, 1978). By being able to rapidly recycle the storage volume several times during a heating season, the utility company has been able to maintain the storage capacity needed for large peak-day deliveries.

Other minor uses of underground space in the Denver area include access tunnels and excavations for commercial space under some of the older large buildings in the downtown area (Price, 1982), and the Foothills Tunnel for the transmission of water from the Stronita Springs diversion dam in the South Platte River Canyon to a treatment plant east of Roxborough Park (Table 5).

ENGINEERING GEOLOGIC PRACTICE IN DENVER

The need for engineering geology appeared early in the history of Denver. Precious metals mining and subsequent railroad and highway construction into and from the Denver area, which served as a trade, transportation, and processing center, necessitated technical analyses of mining engineering and route feasibility. The aridity of the Denver area also required extensive irrigation and water-supply engineering early in its history, to serve agricultural and domestic water needs.

The need and demand for engineering geology in the Denver area has continued to expand in proportion to the rapid economic and social growth of the region. The first legislation (Shelton and Prouty, 1979) with significant impact on the practice of engineering geology in Colorado was Senate Bill 35, passed in 1972, which dealt with land subdivisions. The bill requires reports on the geologic characteristics significantly affecting the proposed land use for all new subdivisions in unincorporated areas of the state. A *subdivision* of land is defined in the law as any division of land into parcels of 35 a (14.2 ha) or less. Since enactment of Senate Bill 35, most geologic reports required by the law have been prepared by engineering-geologic consultants for private subdivisions and/or land developers. Reports are required to be submitted to county planning departments which, in turn, submit them to the Colorado Geological Survey for review and comment. Approval or disapproval of a subdivision is a county-government decision. The State Geological Survey has no regulatory authority over a county-government decision based on the geologic report.

House Bill 1574, passed in 1974, requires that all geologic reports prepared for governmental review must be prepared by a professional geologist. A professional geologist is currently defined as an individual with at least 30 semester hours of geological education and five years of experience.

Two attempts have been made to enact a geologist-registration law in Colorado; both have been unsuccessful. The first attempt, in 1973, provided for registration of both geologists and geophysicists, and each class would have been examined separately. A "grandfather clause" and reciprocity with other states was included with this legislation. The second registration bill, introduced in the 1976 legislative session, was restricted to the registration of engineering geologists. The provisions of this bill were otherwise similar to the first. By limiting the registration requirement to engineering geologists, it was felt by the proponents of this bill that it would stand a better chance of passage, as much of the opposition to the first bill came from outside the engineering-geology profession. Today geologists may register as "Professional Engineers," but this requires an undergraduate engineering education and successful completion of "Engineer-in-training," and "Professional Engineer" examinations.

House Bill 1041, passed in 1974, requires the Colorado Geological Survey to assist local governments in identifying and designating geologically hazardous areas subject to avalanches, landslides, rockfalls, mud and debris flows, unstable slopes, seismicity, radioactivity, ground subsidence, expansive soils and rock, and mineral resource areas. The Colorado Geological Survey also helps adopt guidelines for the administration of these special state interest areas (Rogers et al., 1974).

Two other pieces of recent legislation have had a significant impact on the practice of engineering geology in the Denver area and Colorado. House Bill 1529 (the "Sand and Gravel Bill"), passed in 1973, precludes any governmental body in the state from zoning for alternate uses any area of mineral deposits deemed to have significant economic or strategic value. The law applies to any city and/or county having a population of 65,000 or more, and requires local governments to adopt a master plan for the extraction of commercial mineral deposits. However, no penalties are assessed for failure to comply with the master plan requirement. Most of the emphasis of House Bill 1529 is directed toward production of aggregate and long-range plans for their extraction in the populous (high demand markets) counties of the state. Denver City and County prepared an extraction plan in accordance with the law, but was exempted by the state because of the small area involved.

A companion bill, House Bill 1065, passed in 1973 (Colorado Mined Land Reclamation Act), established a mined land reclamation board with a mandate to ensure proper reclamation of mined-out areas in the state. The result of legislation passed since 1972 is that engineering geology is in great demand in the Denver metropolitan area, and will continue to experience great demands in the future. Between 1970 and 1980 the population of the metropolitan area increased by 30 percent, and housing units by 57 percent. The Denver Regional Council of Governments predicts that the population of the metropolitan area will increase to 2.4 million by the year 2000.

Although the City and County of Denver does not have a city geologist, several adjacent suburbs and counties do have geologists or planners with geology backgrounds on their staffs. Sand and gravel, crushed-rock aggregate, and clay products companies in Denver employ engineering geologists for exploration, development, and reclamation. Private consulting companies are also busy preparing subdivision reports, reclamation plans, hazard assessments, and geotechnical designs for the continued rapid growth of the Denver metropolitan area.

ADDITIONAL INFORMATION

For those people who are interested in seeing some of the engineering geology conditions and situations described in this report, several published field trip logs for the Denver area exist. The references can be found in the bibliography under Weimer and Haun, 1960; Conference Field Trip Committee, 1969; Hansen et al., 1976; and Kirkham, 1981.

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Geology of Boulder, Colorado, United States of America

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Cover photo: Aerial view of the City of Boulder, looking west.

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Geology of Boulder, Colorado, United States of America

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FOREWORD

Boulder—as substantial and heartwarming a name as any geologist could hope for in a city! This name, bearing all of the substance of geology, stands now for a city that has perhaps coaxed all of the maximal benefits from an overbearing geological surrounding, yet has to answer to only a moderate degree of geologic constraint.

Boulder came to be, similar to many New World cities, as a convenient place from which to search for fortune. Prospectors chose this hospitable location because of its decent climate, ready supply of surface water, and accessibility to the nearby mountainous mineral regions. Mid-19th century gold discoveries sealed the fortune of Boulder, which became a town in only one year (1859) and the site of the proposed University of Colorado within another three years.

Though Boulder lies but 43 km northwest of Denver, its larger neighbor (see AEG *Bulletin*, Vol. XIX, No. 3, 1982), it marches to the tune of different drummer. Like Denver, the convenient location between prairie to the east, and great mountains to the west, has the shaped the city's development. Boulder's dependence on these mountains began with its initial use of stream water, and became soundly established with its first of many acquisitions of mountain land for parks (1898) and for water supply (1898). These mountain slopes are now incontrovertably the property of all citizens and will not likely be altered by hillside development. Boulder residents are, by and large, relatively young and are naturally mindful that unbridled growth may not be wholly desirable. These citizens have chosen to regulate the rate at which the city grows.

The surrounding Boulder County, however, has become the population sorption mechanism and is now half-again more populous than the city. Both units have come to realize that cooperation in government is essential. Much of the geologic overtones of growth and development are now managed cooperatively between city and county.

Boulder geology encompasses a variety of rock types and geologic terranes. Precambrian crystalline rocks underlie the western edge of the city which rises abruptly toward the Rocky Mountains. These rocks are in unconformable and fault contact with a gently rolling terrain underlain by tilted sedimentary rocks of upper Paleozoic to upper Mesozoic age.

The city floor lies mainly on Quaternary alluvium and Cretaceous-aged claystone and shale, with interbeds of sandstone and conglomerate. The claystone and shale in this series are commonly expansive. Seven types of surficial deposits mantle the bedrock, the most common of these are a series of weathered pediment alluvial units containing the numerous boulders which gave the city its name.

The majestic mountains are the focus of a good deal of discussion relating to regional seismicity. Rocky Mountain uplift is no longer thought to be an over-and-done-with Laramide event. As the range is now regarded as the northern extension of the Rio Grande rift, this tectonism may well have been reactivated in late Miocene time, and may also have experienced major uplift during Pliocene-Holocene time. These revelations make less comforting the traditional geologic presumption that regional seismicity is essentially of low magnitude. Candidates for *capable* faults have been discovered within the county, and as close as 16 km to Boulder.

Among its geologic blessings, Boulder counts a plentiful supply of sand and gravel. Though over a third of this resource has been compromised by urban development, comprehensive planning and zoning has reached to protect the remaining reserves. This same planning and zoning, among the most advanced east of west-coast America, is exercised at both the city and county level and recognizes the need to regulate resources and to respect geologic constraints and natural hazards. Flood plains are now zoned, but remain subject to disastrous cloudburst discharge from storms centered in the mountains. Planning and/or zoning policies have been extended to wetlands, to areas of slope instability, and to areas of potential soil swell and collapse. Sand and gravel extraction pits are reclaimed as an obligation of regulatory permitting.

There are extraction sites for cement-grade limestone, dimension stone, oil and gas, and some metals within the county. Coal mining, once extensively worked in the Front Range, has ceased as of 1976, leaving significant areas of subsidence hazards near Boulder.

Boulder is one of the relatively few American cities that has developed a full engineering and environmental geologic data base from which to guide its growth and development. Both city and county have benefited enormously from the nearby Colorado Geological Survey (CGS) and the U.S. Geological Survey (USGS). Most of the hazards maps and geological constraints identification, and resources-based zoning measures draw their basic information from published and unpublished data developed by these organizations.

Boulder County concerns for meeting geologic and natural resource constraints additionally deal with coal-mine subsidence, fires in abandoned mines, and mineral extraction. Due to statutory and the USGS influence, a City Geologist (1969–1979) and a County Geologist (1976–1980) were employed in developing a distinct series of unpublished "hazards maps" that are available for use and reference at the respective government offices. The surveys remain in support of the city and county. The Colorado Survey, in particular, devotes part of its small staff to review development plans for the city and county on an as-requested basis.

Geotechnical practice favors west-coast methods, especially in terms of field exploration methods and laboratory testing. Much of this influence relates naturally to the influences of the normally-consolidated nature of most Boulder area soils. Shallow to intermediate depth concrete foundations serve for most structures, which are limited in height to medium rise (seven stories). The city's frequent near-surface ground-water conditions lead to the use of sheet piling for construction excavations. Geotechnical practitioners have been in the forefront of their profession by developing and implementing their own as well as worldwide remedial technology to combat potential soil expansion and collapse damage to engineered structures.

Environmental engineering concerns are dominated by the need to manage solid waste and waste water treatment sludge. Landfill disposal facilities located in the county, have led to some instances of degradation of surface water and ground-water quality. One sanitary landfill is now targeted for Superfund remediation of hazardous wastes co-disposed with municipal refuse.

All bodes well for Boulder, in geologic terms. The city is well informed of its geologic environment; it cannot mentally escape its mountain-back reminder of those influences, it cooperates with its county in management of these resources and constraints, and it has three-sides-room for slow-paced expansion, under pressures of urban development.

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ABSTRACT

Boulder, Colorado is situated in one of the most scenic areas along the Front Range of the Southern Rocky Mountains. The city is located on the western edge of the Denver Basin and is surrounded by the economic mineral deposits that first brought development to the area: gold and silver to the west, coal, oil and gas to the south, east and north. Sand and gravel deposits have been identified and a county wide master plan has been developed for their extraction. In spite of the city's and county's far-sighted and informed leadership, some problems relating to geologic and hydrologic processes have occurred. Marshall Landfill, a solid waste disposal facility, has so seriously contaminated surface and ground water with hazardous waste that it is on the U.S. Environmental Protection Agency's Superfund Cleanup list. The first phase of remedial action taken at this site cost nearly a half million dollars. Much of the city is built within flood plains and significant property damage is expected in the event of a 100-yr or larger flood. Expansive and collapsible soils cause a significant amount of property damage each year although design solutions are well known and effective if constructed properly. Numerous landslides and debris flows have occurred within the city causing damage and are likely to cause more damage in the future. South of the city there are numerous abandoned underground coal mines. Subsidence over these mines has become a problem as housing development moves into the area. Boulder is addressing these geologic and hydrogeologic hazards through city, county, and state programs aimed at identifying and quantifying the hazard and implementing governmental regulation designed to discourage irresponsible land use.

INTRODUCTION

Boulder, Colorado is located 43.4 km (27 mi) northwest of Denver at the base of the foothills of the Rocky Mountains (Figure 1). The population of Boulder in 1985 was approximately 80,000. Since Boulder serves as the hub of activity and as the county seat, this paper includes significant geologic considerations in the county wide area which impact the city's overall growth.

Elevations in Boulder County range from 1,495 m (4,900 ft) above mean sea level to 4,345 m (14,256 ft) at Longs Peak on the Continental Divide, which serves as the county's western border. The City of Boulder is located at elevation 1,629 m (5,340 ft), but elevations increase rapidly to the west in the foothills and beyond. The geographic center of the city is located at 40°00′N latitude and 105°16′W longitude (Figure 1).

Storage Technology, IBM, and the University of Colorado are the major employers in Boulder County, employing over 16,700 people as of 1980 (Boulder Chamber of Commerce, 1980). Boulder is an internationally recognized site for research and development work. It is one of the world centers for research in the atmospheric sciences, hosting facilities for the National Center for Atmospheric Research (NCAR), the National Oceanographic and Atmospheric Administration (NOAA), the Joint Institute for Laboratory Astrophysics and the Environmental Sciences Services Administration Research Laboratory.

History of Founding

On October 15, 1858, Captain Thomas Aikins and about 20 prospectors established a camp at the mouth of Boulder Canyon. They were the nucleus for the first white settlement established in the area. Originally, Captain Aikins had intended to go to the Cherry Creek diggings west of Denver, but when he saw Boulder Valley he decided to settle there. In January 1859, gold was discovered at Gold Run Creek in the mountains west of Boulder. This brought many prospectors into the area. The Gold Hill Mining District quickly became a major gold producing center in the Rocky Mountains. The Boulder City Town Company was founded on February 10, 1859. The settlement was named after the numerous boulders that were and are still present on the area's alluvial surfaces and terraces. They divided the land north of Boulder Creek into 4,044 lots for sale at $1,000 each. Despite the enthusiasm of this first group of real estate investors, $1,000 was a considerable sum of money at the time and many new arrivals chose to continue west into the mountain towns or to settle on farm lands east of Boulder. Prior to the arrival of the white man, Arapahoe Indians roamed and hunted in the area. By the early 1860's, most of the Indians had drifted away as hunting became more difficult with increasing development of the area.

In 1862 Boulder was named as the site of the future University of Colorado. On November 4, 1871, Boulder was incorporated as the "Town of

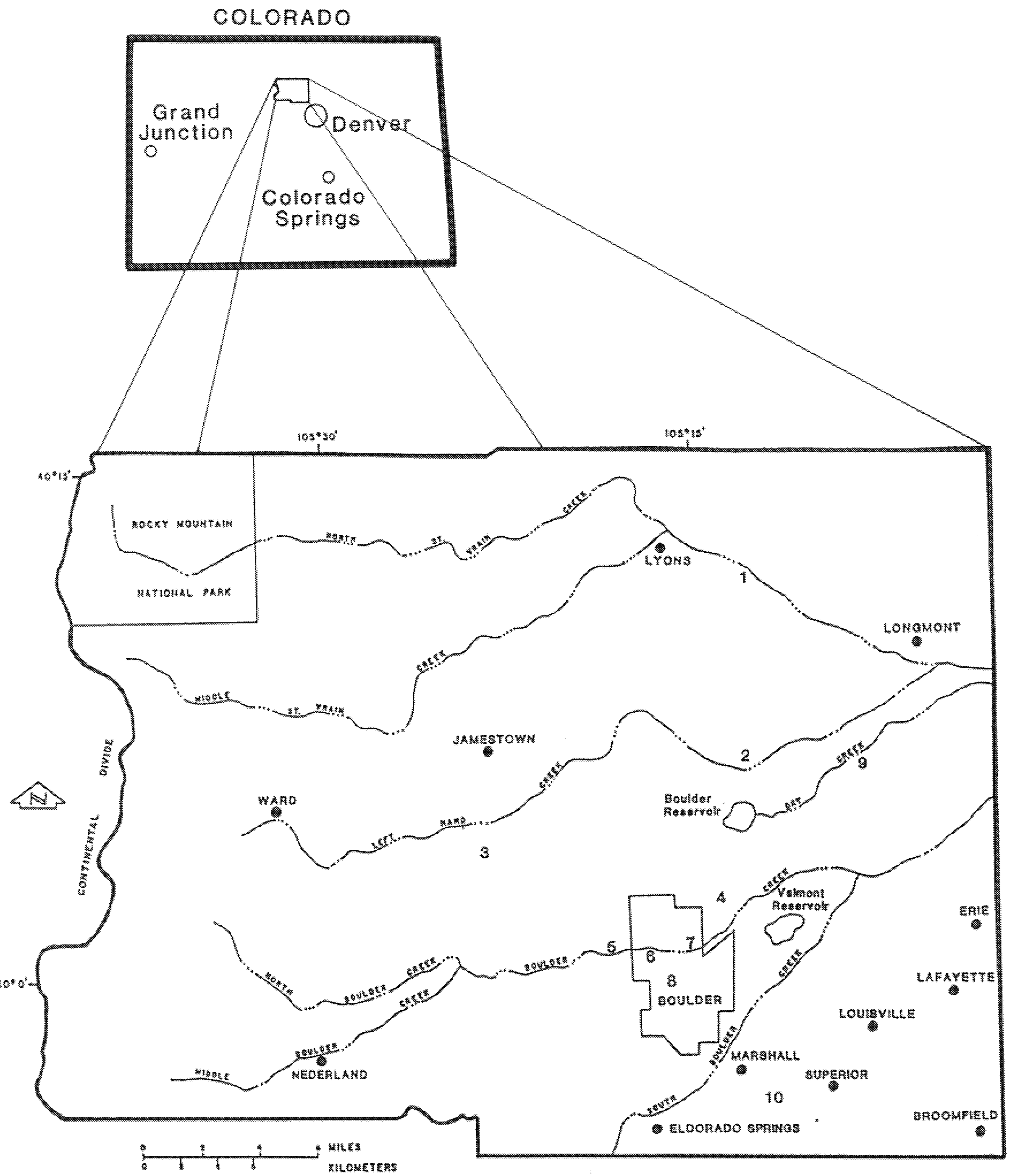


Figure 1. Location map of the City of Boulder, Boulder County, and features in Boulder County mentioned in the text. 1) Martin Marietta limestone quarry and cement plant, 2) Haystack Butte warm water well, 3) Gold Hill mining district, 4) Gunbarrel area, 5) location of large boulder in Boulder Creek in Figure 18, 6) location of municipal building in Figure 19, 7) location of the bank building in Figure 20, 8) University of Colorado campus, 9) location of Walden Ponds Wildlife Habitat and active gravel pits in Figure 15, 10) Marshall landfill.

Boulder." During the next decade the population jumped from just a few hundred to more than 3,000, largely because of the completion of the Colorado Central Railroad. The population in the City of Boulder was 6,000 in 1900, 10,000 in 1910, and about 11,000 in 1920. The present explosive growth began after World War II. In 1976, the City of Boulder made headlines by officially adopting a policy to limit growth to 2 percent per year. This policy is still in effect and is implemented through the allo-

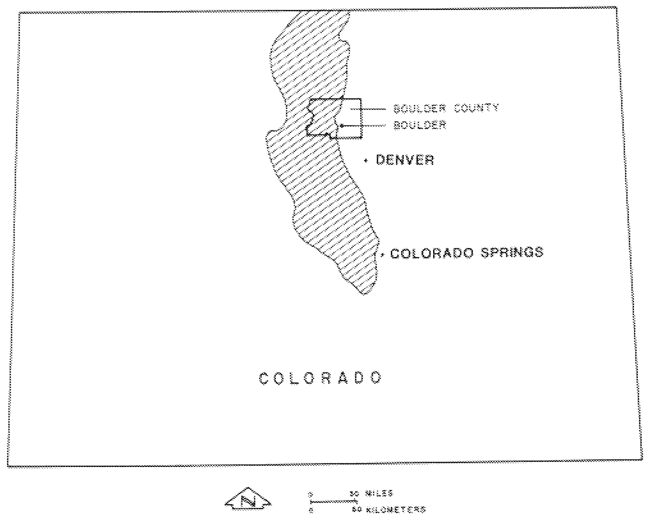


Figure 2. State of Colorado showing the Front Range (ruled) in relation to Boulder and Boulder County.

cation of building permits. The population of Boulder County is now over 200,000 people, with more than 80,000 in the city proper. The population is considered young, over 50 percent of the people are less than 30 years old (Boulder Chamber of Commerce, 1980).

Acquisition of mountain land by the city began in 1898 with the purchase of 80 acres. In 1899 and 1907, Congress granted the city an additional 1,780 acres of land on Green Mountain, Bear Mountain and Flagstaff Mountain. These lands, together with later purchases, comprise a "greenbelt" of public land which surrounds Boulder and guarantees a permanent backdrop of undeveloped mountain scenery.

Climate

Boulder has a semiarid, continental climate. Winters are cold and dry and summers are cool and relatively dry. The climate is characterized by intense sunlight, low humidity, relatively low precipitation, occasional high winds and large temperature fluctuations. The annual mean temperature is 10.4°C (50.7°F), with a mean summer temperature of 18.7°C (65.6°F) and a mean winter temperature of 3.4°C (38.1°F). The annual precipitation is 46 cm (18.1 in.), most of which falls in early spring. The prevailing winds at Boulder blow in from the west. These westerly winds are profoundly influenced by the topography provided by the eastern flank of the Rocky Mountains. Cold dense air flows over the crest of the mountain range and accelerates and warms as it flows down the eastern side. The valleys act as vortices that channel the wind and cause even greater velocities. Portions of the city have experienced gusts of winds in excess of 193 km/hr (120 mi/hr). Seasonally, these "chinook" winds have caused damage to building roofs and windows.

GEOLOGIC SETTING

The City of Boulder is located at the eastern edge of the Front Range, the easternmost range of the Southern Rocky Mountains (Figure 2). This location also places Boulder at the western edge of the Denver Basin, one of the largest structural foreland basins in the Rocky Mountains. Originally, the city was built at the mouth of Boulder Canyon where Boulder Creek leaves the Front Range and enters the Colorado Piedmont Section of the Great Plains. Since it was founded, Boulder has expanded only a short distance westward into the mountains, with most of its growth eastward across the plains. Thus, the westernmost part of the city is underlain by Precambrian igneous and metamorphic rocks, while the greater part of Boulder is underlain by upturned late Paleozoic and Mesozoic sedimentary rocks that dip up to 50° east (Figures 3, 4 and 5), and are partly covered by a thin layer of Quaternary surficial deposits (Wrucke and Wilson, 1967; Wells, 1967).

Just 32 km (20 mi) due west of Boulder, the Continental Divide lies along the crest of the Front Range, with elevations reaching 4,115 m (13,500 ft) in the Indian Peaks Wilderness Area south of Rocky Mountain National Park. Longs Peak, at 4,345 m (14,256 ft), is the highest point in the county and is

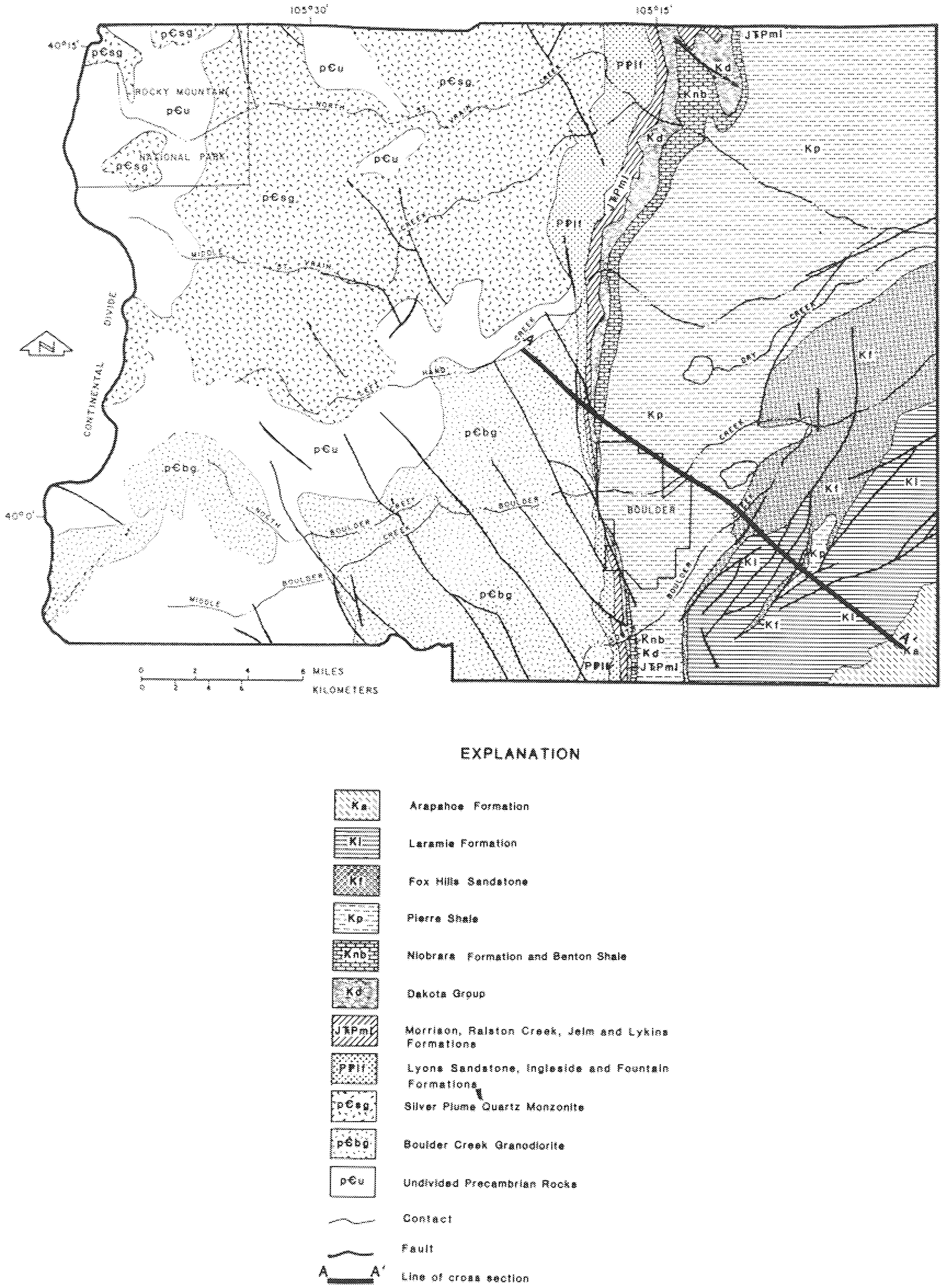


Figure 3. Generalized bedrock geologic map of Boulder County, adapted from Wrucke and Wilson (1967), Wells (1967), Trimble (1975), and Hall and others (1980).

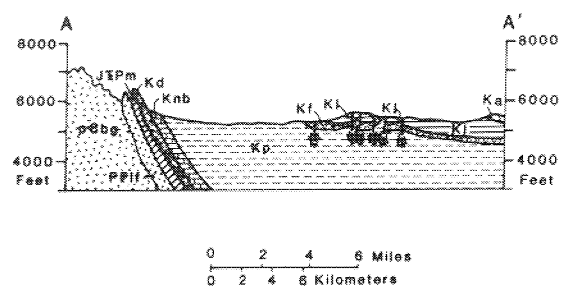


Figure 4. Generalized NW-SE geologic cross section across Boulder. Location shown as A-A' on Figure 3. See Figure 3 for explanation of symbols. Adapted from Hall and others (1980).

located in the extreme northwest corner of the Park. This part of the Front Range is composed primarily of Precambrian granitic rocks intruded into an older suite of metasedimentary and metavolcanic rocks. Cutting this Precambrian terrane is the Colorado Mineral Belt, a narrow northeast-trending zone that contains most of the major mining districts of Colorado. The belt extends from Durango, Colorado in the southwest to the Boulder area in the northeast. The mineralization in the belt is related to stocks, dikes and sills of Lake Cretaceous-Early Tertiary (Laramide), and middle Tertiary age that intruded a major northeast-trending Precambrian shear zone (Tweto and Sims, 1963; Romberger, 1980). Structural elements of this Precambrian shear zone are thought to project northeastward beneath the Denver Basin and may have been reactivated several times during Phanerozoic time, providing structural traps for hydrocarbon resources discovered in the area. This shear zone crosses the southeastern corner of Boulder County (Figure 3) and may also have played a significant part in localizing the extensive coal deposits of the Boulder-Weld Coal Field (Davis and Weimer, 1976).

The mountains of the present-day Front Range, as well as the entire Southern Rocky Mountains, attained their lofty elevations and dramatic relief in Neogene times and should not be considered Laramide age (Late Cretaceous-Early Tertiary) mountains as they are commonly regarded (Bilodeau, 1987). This misconception has been fostered over the years by the intense study of the pervasive Laramide age deformation, igneous intrusion and mineralization found throughout the region. During the Laramide orogeny there were mountains present in the region, but they had a more subdued topography since erosion and sedimentation kept pace with the uplift of the mountain blocks and subsidence in the basins. By the Late Eocene, a low relief erosion surface was present across the entire region (Epis and Chapin, 1975; Epis et al., 1980; Colman, 1985; and Scott and Taylor, 1986). This erosion surface has been subsequently uplifted and locally broken up throughout the Southern Rocky Mountains by Neogene high-angle normal faulting. This uplift, which is still continuing (Kirkham and Rogers, 1981), and its accompanying erosion, has produced the elevation and relief of the present-day Rocky Mountains (Izett, 1975; Scott, 1975; Taylor, R. B., 1975; Epis et al., 1980; and Trimble, 1980). This uplift is related to the northward propagation and evolution of the Rio Grande Rift, a major Neogene extensional feature of crustal dimensions in central New Mexico and Colorado (Riecker, 1979; Cordell, 1982; and Colman et al., 1985).

The Denver Basin, which underlies Boulder, is a large north-south-trending asymmetric structural basin with a gently dipping east flank, that formed during the Laramide orogeny. The City of Boulder is spread out across the eroded and truncated edge of a steeply east-dipping stratigraphic section of middle Pennsylvanian through Late Jurassic nonmarine clastic sedimentary rocks overlain by Cretaceous marine deposits about 3,640 m (11,942 ft) thick (Figure 4). Along the mountain front, from the mouth of Boulder Creek southward, several formations and over 450 m (1,476 ft) of section have been faulted out along the north-northwest-trending Boulder Fault (Wrucke and Wilson, 1967; Wells, 1967). No Laramide age sedimentary rocks are preserved within the city limits of Boulder, though they are present only 8 km (5 mi) to the southeast, near Broomfield.

Bedrock Geology and Geologic History

Precambrian crystalline rocks make up most of the western half of Boulder County and underlie the westernmost fringe of the City of Boulder itself (Figures 3 and 5). These Precambrian rocks consist of folded metasedimentary and metavolcanic gneisses and schists about 1,800 m.y. old that were intruded twice by Precambrian rocks: the Boulder Creek Granodiorite, about 1,700 m.y. old; and the Silver Plume Granite about 1,400 m.y. old. Precambrian fault trends are predominantly north-northwest with a subsidiary set of faults and shear zones that trend northeast. These faults have been reactivated repeatedly throughout Phanerozoic time (Wells, 1967; Tweto, 1980).

The lower Paleozoic section, Cambrian through

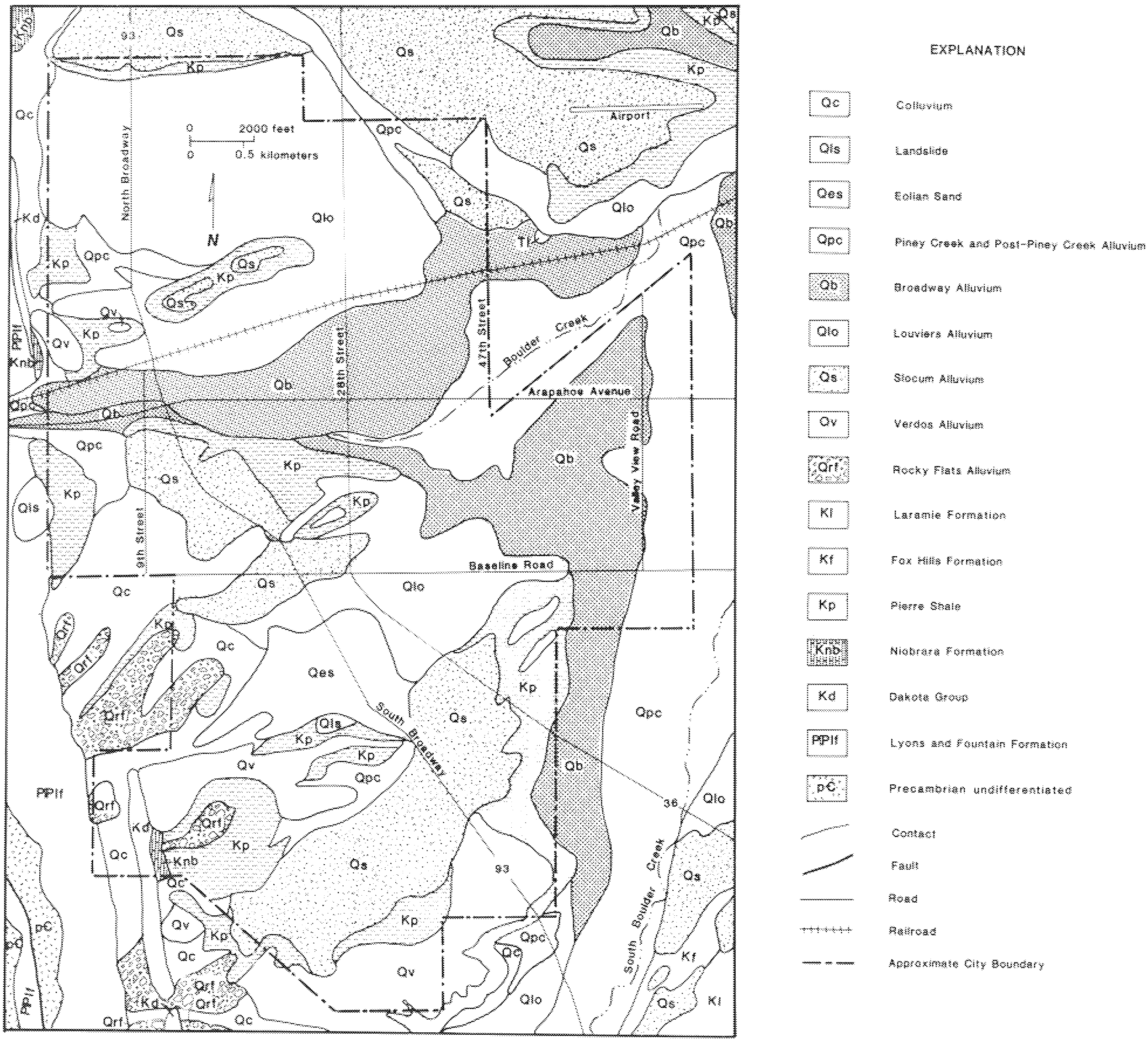


Figure 5. Geologic map of the City of Boulder. Adapted from Spencer (1961), Wrucke and Wilson (1967), Wells (1967), Trimble (1975), Colton (1978), and Trimble and Machette (1979).

Mississippian, is missing in the Boulder area (Figure 6). Here, Pennsylvanian rocks rest unconformably on Precambrian basement. Although the closest lower Paleozoic rocks are on the western side of the Front Range and about 80 km (50 mi) to the south, beneath the Denver Basin, it is thought that lower Paleozoic marine rocks once covered the Precambrian terrane in the area (Ross and Tweto, 1980). Blocks of Cambrian through Devonian age limestone and sandstone have been discovered in a series of diatremes 100 km (62 mi) to the north at the Colorado-Wyoming border, in an area where no lower Paleozoic strata are known (Chronic et al., 1969). This suggests that early Paleozoic marine seas transgressed across the region from time to time. By Early Pennsylvanian time, the Boulder area was uplifted and became situated on the northeastern flank of the Ancestral Front Range uplift, one of several northwest-trending mountain ranges that comprised the late Paleozoic Ancestral Rocky Mountains (DeVoto, 1980; Maughan, 1980). This mountain building episode was the result of reactivation of Precambrian basement structures caused by the continent-continent collision in progress to the southeast. This plate collision, involving the African-South American plate and the southern mar-

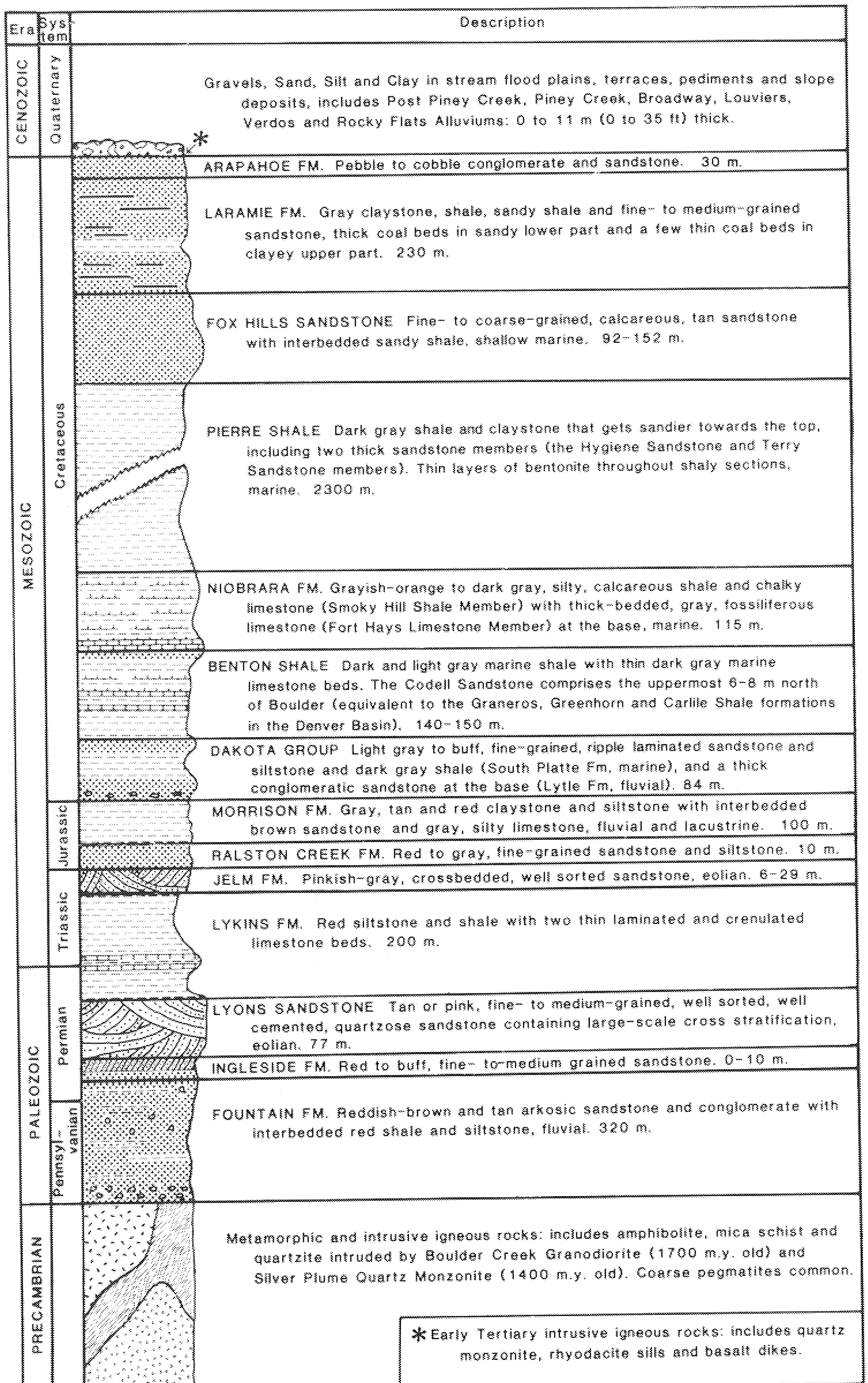


Figure 6. Stratigraphic column for Boulder County. Adapted from Malde, (1955), Wells (1967), Wrucke and Wilson (1967), and Madole (1973).

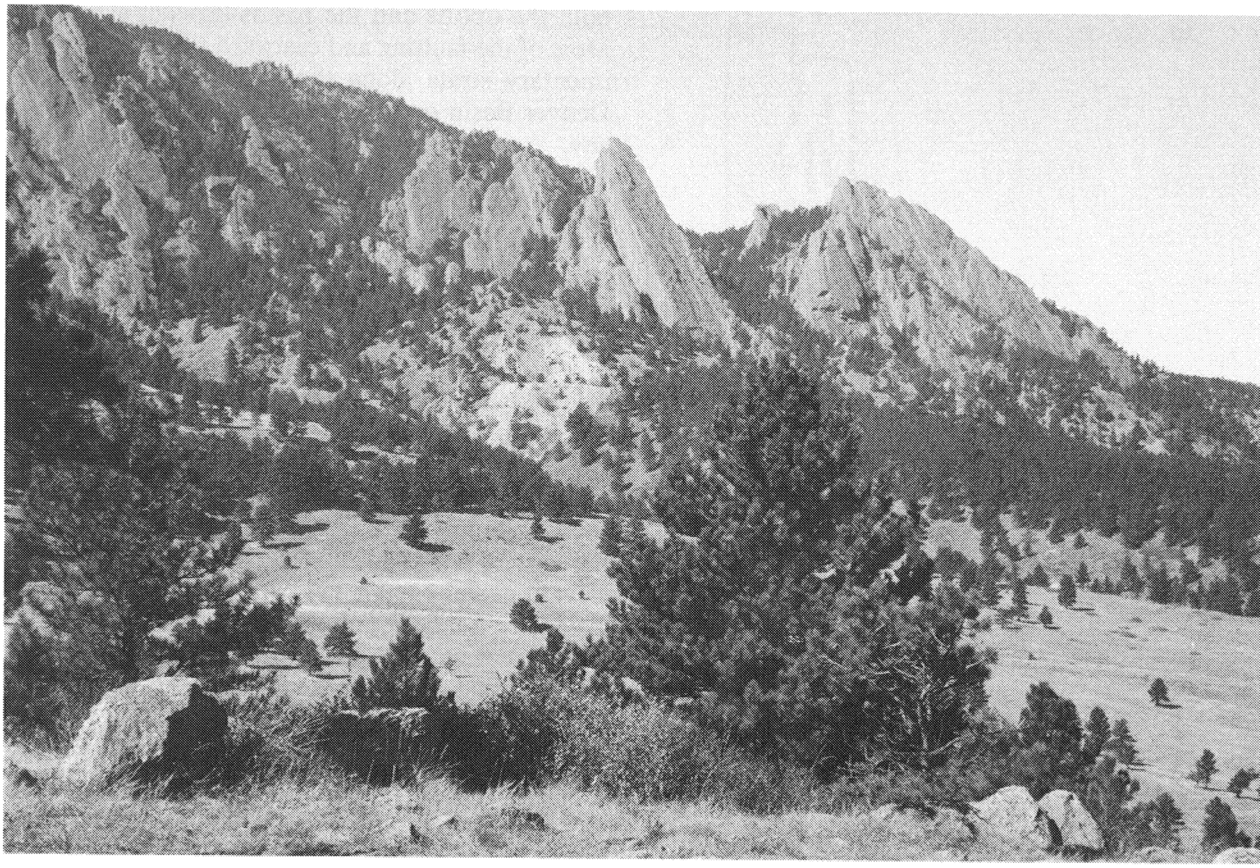
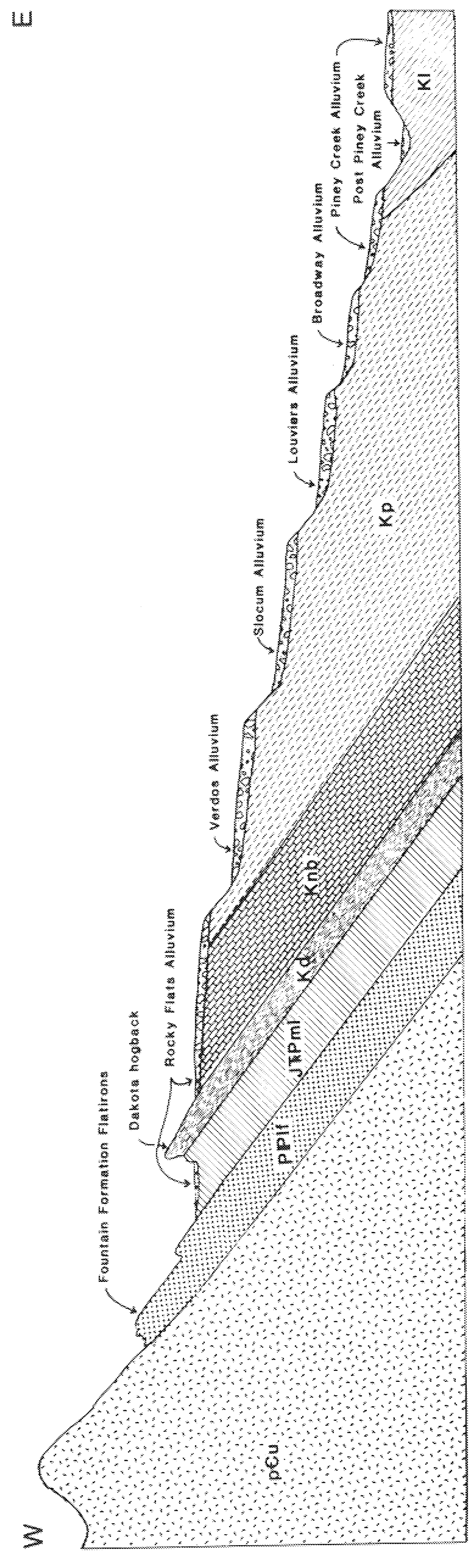


Figure 7. Flatirons of the Fountain Formation just southwest of downtown Boulder.

gin of the North American plate (Kluth and Coney, 1981), has been termed the Ouachita orogeny. Uplift of the Ancestral Front Range caused erosion of the lower Paleozoic section and deposition of the conglomerate and arkose of the mid-Pennsylvanian Fountain Formation. This formation makes up the Flatirons, the prominent rock formations for which Boulder is famous (Figure 7). By the end of the Paleozoic, the Ancestral Front Range was a low-standing set of hills with eolian sandstone, finer-grained nonmarine and marginal marine redbed clastics being deposited in the area.

In the Early Cretaceous, the entire region began to subside due to crustal loading to the west in the developing backarc Sevier fold and thrust belt (Jordan, 1981). A major marine transgression into this subsiding foreland basin produced the intracontinental Cretaceous Interior Seaway (Kauffman, 1977). Both the Ancestral Front Range and the Denver Basin area became buried under about 3,000 m (9,842 ft) of marine and marginal marine strata of Cretaceous age (Figure 6). Most excavations within the City of Boulder will encounter the Late Cretaceous Pierre Shale as bedrock beneath thin Quaternary alluvium (Figure 5). With a thickness of around 2,300 m (7,546 ft), this marine shale is the thickest stratigraphic unit in the Boulder area (Figure 6).

At the end of the Cretaceous Period, this large foreland basin began to break up with the formation of basement-cored, thrust and reverse fault-bounded uplifts and smaller intermontane basins. This Late Cretaceous-Early Tertiary tectonic episode is called the Laramide orogeny. Beginning about 67.5 m.y. ago (Tweto, 1975), a mountain range with much the same configuration as the present Front Range was uplifted, contributing to the formation of the Denver Basin to the east (Bilodeau, 1987). Erosion of sedimentary strata from the rising Laramide Front Range kept pace with the rate of uplift and deposition in adjacent basins so that the actual topographic relief of the mountains was never great, much less than in the Rockies of today. By the Late Eocene, uplift had ceased and a regional, low-relief erosion surface was developed across the area of



both the uplifts and the basins (Epis et al., 1980). Most of the faulting and eastward tilting of the sedimentary strata along the western margin of the Denver Basin occurred during the Laramide orogeny.

During the Paleocene, rhyodacitic sills and basaltic dikes were intruded into the sedimentary section in the Boulder area. Larson and Hoblitt (1973) report a radiometric age of 64.6 ± 2.4 m.y. on the Flagstaff mountain sill on the west side of the city and consider the east-northeast-trending Valmont dike on the east side of Boulder to be of similar age. The dike is 6–20 m (20–60 ft) wide, nearly vertical and has been traced, in the subsurface, for nearly 8 km (5 mi) with magnetic surveys (Colton, 1978).

During the Oligocene, the regional topography is thought to have been very similar to the present-day eastern plains of Colorado but with a maximum elevation of about 900 m (2,920 ft) (Trimble, 1980). This setting existed well into the Miocene when major thermal uplift and east-west extension began in central New Mexico and south-central Colorado and formed the Rio Grande Rift (Riecker, 1979). Late Miocene to present regional uplift involving much vertical movement on north-trending normal faults with attendant erosion has produced the Front Range that exists today west of Boulder (Epis et al., 1980). No Cenozoic sedimentary rocks are found in the Boulder area except for the thin Quaternary pediment gravels that blanket the eastern marginal areas of the Front Range.

Surficial Deposits

In the Boulder area, bedrock crops out at the surface or is locally covered by thin Quaternary surficial deposits. The Quaternary deposits are of seven main types: 1) glacial and periglacial deposits, 2) residual regolith, 3) pediment and terraced alluvium, 4) slope wash colluvium, 5) talus and landslide deposits, 6) valley fill alluvium and 7) eolian deposits (Wells, 1967; Wrucke and Wilson, 1967; and Madole, 1973). The mountainous western part of this area has been subject to continuous weathering and erosion since uplift began in Early Tertiary time, with the most recent episode of renewed uplift in the Pliocene or, possibly, continuing today (Scott, 1975). From the Continental Divide to about 16 km (10 mi) west of

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Figure 8. Schematic cross section showing relative vertical relationships of the pediment and terraced alluvial units. Symbols as on Figure 3. Adapted from Scott (1963). Not to scale.

Boulder, glacial till and periglacial deposits such as talus, rock glaciers and solifluction lobes, are the main Quaternary units (Madole, 1973). Within the foothills area, a thin mantle of residual regolith overlying deeply weathered metamorphic or granitic bedrock is the dominant type of surficial deposit. In most upland areas, these deposits are only a few centimeters to 1–2 meters in thickness. This small thickness creates problems with respect to water supply and sewage treatment associated with housing development. In many areas, highly weathered Boulder Creek granodiorite is commonly treated as soil because it is easily excavated with a backhoe (Madole, 1973).

All of the pediment and terraced alluvium is of Pleistocene age and is generally divided into five different units: the Rocky Flats, Verdos, Slocum, Louviers and Broadway alluviums (Figures 5 and 8) (Scott, 1960). The first three are pediment gravels; and the last two are valley fill and terrace deposits restricted to present day stream drainages and involve no pediment cutting. These deposits have been correlated to geologic-climatic stages or cycles of Pleistocene glaciation in the higher Rocky Mountains to the west.

The Rocky Flats alluvium is the coarsest and most prominently displayed pediment alluvium in the Boulder area. An older, higher, but locally restricted "pre-Rocky Flats alluvium" is found just north of Coal Creek Canyon, about 13 km (8 mi) south of Boulder, but is not present within the city. From the southern margins of the city, the Rocky Flats alluvium continues southward along the mountain front for tens of kilometers, intermittently interrupted and dissected by major stream drainages. This alluvium is the remnant of an alluvial fan which has its apex at Coal Creek Canyon (Wells, 1967). Lithologically, the alluvium is composed of poorly sorted boulders, cobbles, pebbles and sand in a yellowish brown to red clayey matrix with layers of clay, silt, and sand and is 4.6–11 m (15–35 ft) thick. The grain size of the larger clasts decreases eastward away from the mountain front (Wells, 1967). Rocky Flats alluvium is of Nebraskan or Aftonian age, or about 1.0–2.0 m.y. old, and is about 91–105 m (300–345 ft) above modern stream levels (Scott, 1975; Trimble, 1975). In the southwestern part of the city, the Rocky Flats alluvium caps many of the wooded mesas that project eastward from the mountain front.

The Verdos alluvium (Scott, 1960) is the pediment alluvial deposit found 15–30 m (50–100 ft) below the Rocky Flats alluvium and 61–76 m (200–250 ft) above present streams. It consists of fairly well stratified brown sand and gravel (partly decomposed pebbles, cobbles and boulders of igneous, metamorphic and sedimentary rocks) in a clayey matrix (Wells, 1967; Wrucke and Wilson, 1967; and Trimble, 1975). The Verdos alluvium can be as thick as 10 m (33 ft) but averages about 4.6 m (15 ft). A locally diagnostic white, rhyolitic, volcanic ash bed near the base of the unit has been found at several localities in the region. This ash bed has been correlated by Scott (1960) with the Pearlette Ash Member of the Sappa Formation in Kansas and Nebraska, of Kansan or Yarmouth age or about 600,000 years old. The most extensive deposits occur as gravel-capped alluvial terraces or "mesas" along the Fourmile Canyon Creek drainage at the northern edge of the city and near the middle of town, where the Verdos underlies a major portion of the University of Colorado campus (Wrucke and Wilson, 1967).

The Slocum alluvium is the lowermost pediment alluvium in the area, occurring 15–30 m (50–100 ft) below the Verdos and about 24–40 m (80–130 ft) above present streams. The alluvium is generally similar to the Verdos in lithology and texture, but it is finer grained than the older alluviums (Scott, 1960; Wells, 1967; Trimble, 1975; and Costa and Bilodeau, 1982). The Slocum is generally less than 9 m (30 ft) thick, averaging 6–7 m (20–30 ft) and is considered to be of Illinoian age, or about 150,000 to 260,000 years old (Scott, 1960).

These three upper level pediment alluviums were all deposited by east-flowing streams on steam-cut bedrock surfaces, and all have well-developed soil profiles at their tops. The gravels in these deposits are commonly found coated and cemented with calcium carbonate, and they are oxidized reddish brown throughout, even where they are thick (Scott, 1960; Trimble, 1975). The deposits are thicker in paleochannels and thinner in interchannel areas, with the maximum size and roundness of the boulders and cobbles varying with the source stream and distance from the mountain front (Madole, 1973). Identification of the different alluvial terraces within a single drainage area is based primarily on relative topographic position, with the highest terrace always the oldest, reflecting continued uplift and base level lowering throughout Quaternary time.

The Louviers alluvium is a Late Pleistocene (Late Illinoian age-Bull Lake Glaciation, or about 140,000 years old (Pierce et al., 1976)) valley fill alluvium that is most often preserved below stream terraces along present day east-flowing streams. It consists of slightly weathered, fairly well sorted and stratified, red to yellowish-brown sand, pebbles and cobbles in a clayey silt to sandy matrix. A strong, dark brown, clayey soil has developed at its top (Malde, 1955; Trimble, 1975). The alluvium is composed of two main facies, a coarser gravel facies present along major streams and a finer-grained silty facies that occurs along minor streams (Malde, 1955; Scott, 1960). The limited weathering of the gravel clasts in this unit has made it the major source of commercial sand and gravel in the Boulder-Denver region (Costa and Bilodeau, 1982). The Louviers alluvium has a highly variable thickness of 1–6 m (3–20 ft), occurs about 12–30 m (40–100 ft) below the Slocum alluvium and 6–20 m (20–65 ft) above modern streams (Malde, 1955; Scott, 1960; and Trimble, 1975). The depth of downcutting and base level lowering associated with the Louviers is the deepest of any of the Quaternary alluvial deposits (Scott, 1975; Trimble, 1975). Locally, along minor streams, post-Louviers erosion and downcutting has not cut entirely through the alluvium and modern streams are reworking the Louviers alluvium. Between Boulder Creek and South Boulder Creek, the Louviers alluvium underlies a thin layer of Broadway alluvium 61–183 cm (2–6 ft) thick (Trimble, 1975).

The Broadway alluvium is the lowest Pleistocene (Wisconsin age-Pinedale Glaciation, or about 30,000 years old (Pierce et al., 1976)) valley fill-terraced alluvium in the region (Scott, 1960). Lithologically, it consists of yellowish-orange to reddish brown, cobbly pebble gravel of predominantly Precambrian crystalline rock composition. A poorly developed brown soil is present at its top. The thickness of the alluvium is from 0–9 m (0–30 ft) underlying terraces 6–12 m (20–40 ft) above present streams (Trimble, 1975). To the southeast in the Denver area, the alluvium forms a broad, well-defined terrace upon which the largest and tallest buildings of downtown Denver have been built (Costa and Bilodeau, 1982).

Colluvium, talus and landslide deposits are locally very important but are usually restricted to fairly steep slopes or the bases of steep slopes or cliffs in the mountainous western part of the area. Landslide deposits are discussed in the Geologic Constraints section of this paper.

Valley fill alluvium is the term applied to the Holocene alluviums that fill the modern stream valleys of the area. Two alluvial deposits have been recognized, the Piney Creek alluvium and the post-Piney Creek alluvium (Malde, 1955). The Piney Creek alluvium is composed of brownish-gray silt, sand and clay with interstratified humic-rich layers. Close to the mountain front, gravel as coarse as small boulder size (36 cm or 14 in.) occurs in lenses near the base. The deposit forms terraces 1.2–6 m (4–20 ft) above modern streams and is from 0–6 m (0–20 ft) thick (Trimble, 1975). Carbon-14 dating establishes an approximate age of about 2,800 yrs (Scott, 1963). Post-Piney Creek alluvium consists of grayish-brown, humic, fine sand and silt, with loosely consolidated pebble and cobble lenses near the base. The alluvium is from 0.5–6 m (1.6–20 ft) thick, increases in thickness downstream, and covers almost the entire flood plain of the modern streams. Post-Piney Creek alluvium is primarily derived from Piney Creek alluvium and exhibits little or no soil development (Costa and Bilodeau, 1982). Carbon-14 dating places it at about 1,500 years old (Scott, 1963).

Eolian sand and silt (loess) of Holocene to Pleistocene (Sangamon) age is derived from the older Pleistocene alluviums and is the most extensive surficial deposit in the plains area east of Boulder (Colton, 1978; Trimble and Machette, 1979). This wind-blown deposit is 1–7.6 m (3–25 ft) thick and has a brown Holocene soil developed in its upper part (Trimble, 1975).

Seismicity

Colorado has, in general, been considered an area of low seismic activity. A few moderate earthquakes have caused damage over the past 110 years and hundreds of smaller earthquakes have been instrumentally recorded within the state (Figure 9). No surface fault ruptures have been detected in the state as a result of historic earthquake activity. Recent geological investigations, however, have discovered several faults that are considered "active" and capable of generating earthquakes. There are 5 faults within 80 km (50 mi) of Boulder which exhibit Quaternary (1.6 m.y. old) movement (Figure 10) (Kirkham and Rogers, 1981). The Valmont fault, which is less than 16 km (10 mi) from Boulder, offsets gravels within the Slocum alluvium (0.15–0.26 m.y. old). When evaluating the potential seismicity of this area, it is important to remember that the 4,300 m (14,000 ft) high mountains west of the city were uplifted in Neogene time and that the major uplifts occurred within the last 5 million years (Epis et al., 1980).

Concern about the potential seismicity of the Front Range area was raised in the 1960's when a series

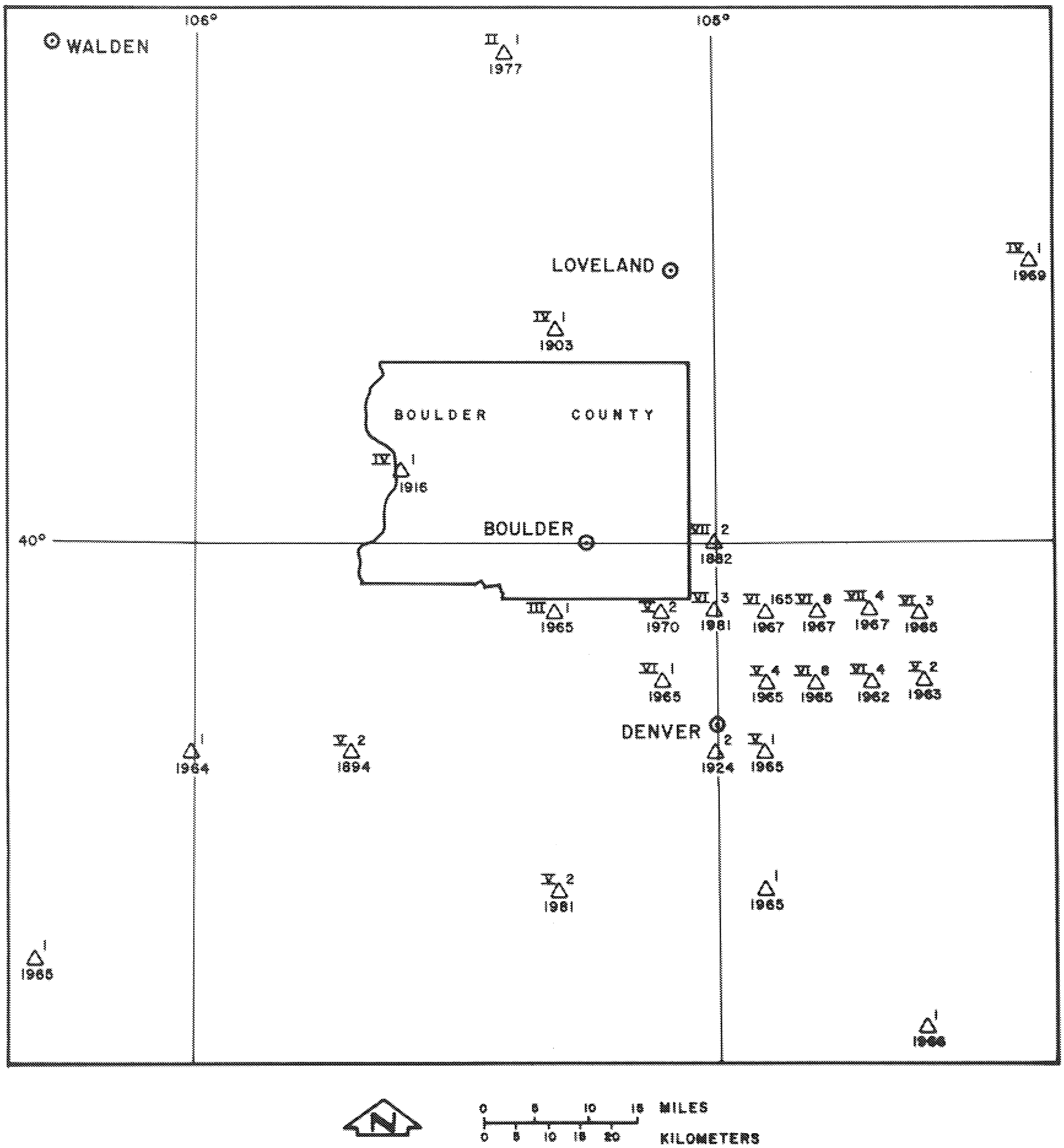


Figure 9. Earthquake epicenters and intensities in the Boulder area and vicinity 1882–1981. The triangles represent epicenters, the number of earthquakes at each location is shown by the number to the right of a triangle. The Roman numeral to the left of a triangle is the maximum Modified Mercalli intensity of all the earthquakes at that location. The number below a triangle is the latest year for which the maximum intensity was recorded. The 1882 earthquake epicenter is shown near Broomfield although other possible locations have been suggested (from Stover et al., 1984).

of earthquakes occurred in Denver, 45 km (28 mi) southeast of Boulder. The series lasted from 1962 to 1968, with the largest earthquake occurring in 1967 and measuring 5.3 on the Richter Scale (Kirkham and Rogers, 1981). The accompanying seismic shaking caused considerable damage in the surrounding suburbs, but Boulder was undamaged. Many geologists who have studied these earthquakes believe that a deep, high-pressure injection well, drilled by the U.S. Army at the Rocky Mountain Arsenal in northeast Denver, triggered the earthquakes (Evans, 1966; Healy et al., 1968). There is evidence that tectonic stresses existed in the area prior to the fluid injection because earthquakes continue to occur even though the well was decommissioned early in 1966 (Kirkham and Rogers, 1981).

The largest earthquake recorded in the area oc-

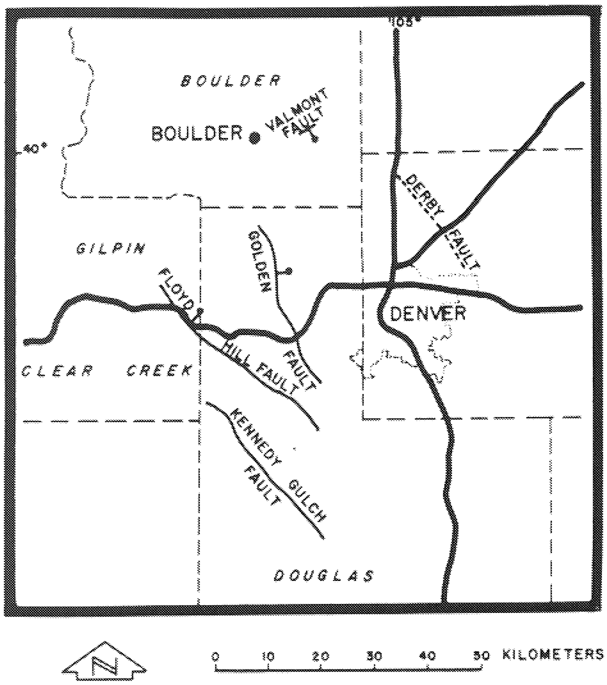


Figure 10. Map showing traces of known Quaternary faults in the Boulder area. Bar and ball are on the downthrown side. Dashed fault is deeply buried and inferred (from Hansen and Crosby, 1982).

curred in 1882. This earthquake caused Modified Mercali shaking of Intensity VII from Denver to Southern Wyoming (Epis et al., 1980). The walls of the Boulder County railroad depot cracked and plaster fell from walls at the University of Colorado. Early accounts of the earthquake suggest it was centered in the Broomfield-Louisville area (Figure 9); however, a Dames and Moore, Inc. (1981) study suggests that the epicenter may have been located in northwestern Colorado.

GEOTHECHNICAL CHARACTERISTICS

The geotechnical characteristics of the surficial and bedrock units in the Boulder area can change dramatically over very short distances. The following general descriptions provide typical conditions that can be expected for specific rock units. Many geologic constraints can be controlled through sound engineering design. On the other hand, areas affected by serious geologic constraints, such as subsidence over abandoned coal mines, are best avoided if possible.

Foundation Related Geologic Units

The valley fill and terraced alluviums consist of boulders, cobbles, gravel, sand, silt and clay. These deposits generally provide sufficient foundation support below a depth of about 0.7 m (2 ft). Locally, there are lenses of expansive clay and/or compressible silts present. These lenses average from 03.3–1.6 m (1–5 ft) thick and can cover as much as a few acres (Gardner, 1968). A caliche (hardpan) layer up to 1.6 m (5 ft) thick commonly is present within 1.0 m (3.3 ft) of the surface. Lenses of organic material 0.3–1.6 m (1–5 ft) thick may also be encountered. Settlement problems may be encountered when structures are constructed over compressible silts or organic-rich materials. Heaving problems may be encountered in areas of swelling clays. Mass movements can occur in valley fill and terraced alluviums. Areas of potential mass movement are generally restricted to hillslopes or along benches which have been undercut. Undercutting can be caused by natural erosion, such as fluvial action or human-induced causes, such as construction. These deposits are generally easy to excavate.

The Arapahoe Formation covers less than 9 km^2 (3.5 mi^2) in the southeast corner of Boulder County. It is composed of conglomerate, sandstone, shale and claystone. Generally, this formation provides adequate bearing strength for most structures. The claystone, however, is often expansive and can swell if moisture is introduced. Shale and claystone are usually easy to excavate, while conglomerate and sandstone may require ripping if they are cemented.

The Laramie Formation consists of interbedded shale, siltstone and sandstone that contain lenses of coal and clay. Locally, the formation is faulted and fractured. The coal in this formation was extensively mined in southeastern Boulder County and subsidence over abandoned mines is a continuing problem. Slope stability is generally good in unsaturated slopes of less than 25°. Numerous slope failures have occurred in the southern portion of the county where slopes exceed 25° and where bedding surfaces or clay seams dip in the same direction as the hillslope but at a smaller angle relative to the slope (Figure 11). Foundation suitability of this formation is generally good where expansive clay, slope stability and subsidence problems are absent. Excavation of the shale and siltstone beds is relatively easy using conventional methods. Excavation of the sandstone beds is moderately difficult.

The Fox Hills Sandstone is composed predominantly of sandstone and siltstone, which vary from unconsolidated to very hard. This formation generally has high bearing strength. Excavation of the siltstone beds is relatively easy, but where cemented

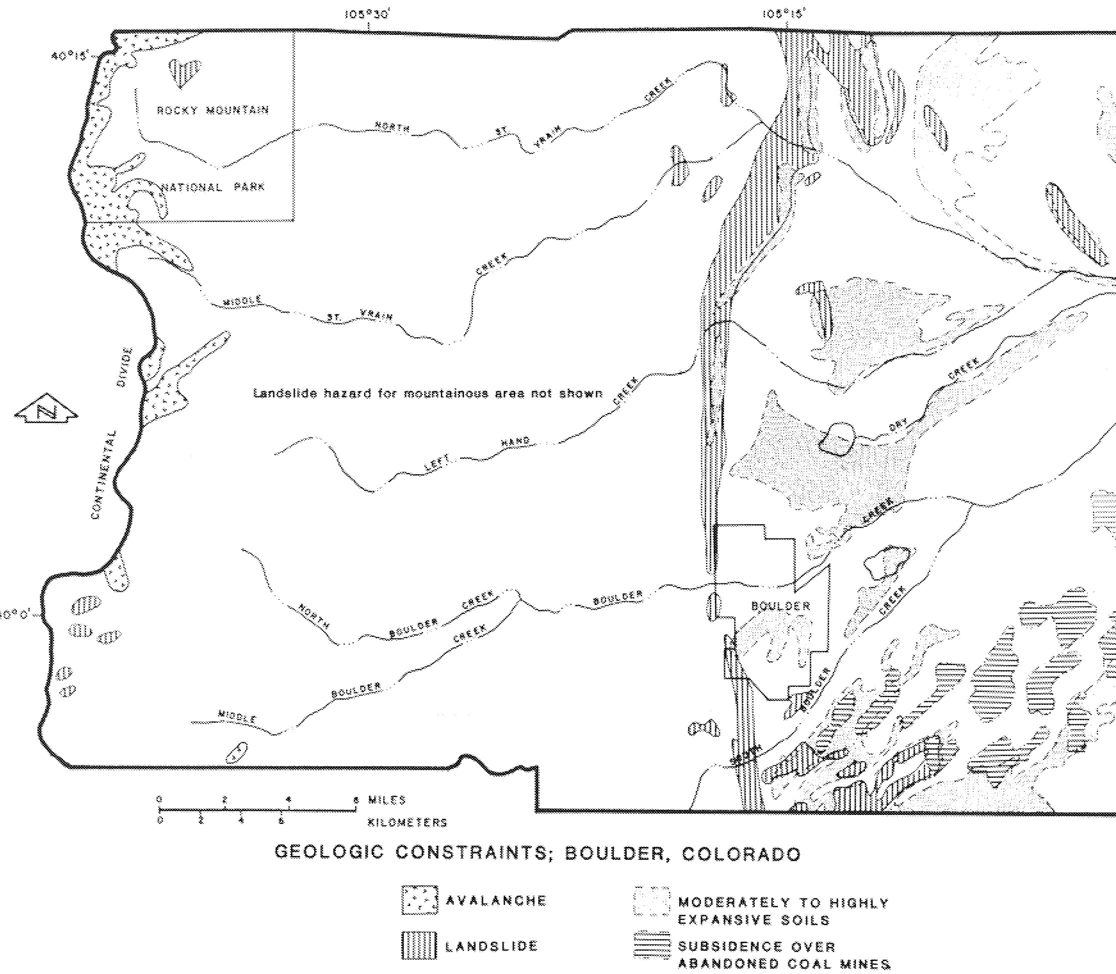


Figure 11. Geologic constraints map, Boulder County. Modified from Madole (1973), Hart (1974), and Boulder County Planning Commission (1978).

sandstone beds are encountered, blasting may be required for foundation excavations.

The Pierre Shale consists of 2,300 m (7,546 ft) of shale with thin layers of bentonite, siltstone and sandstone. The strike of the formation if roughly north-south and the dip is nearly vertical adjacent to the mountain front. The dip flattens to 25° where the formation crops out on the eastern side of town. This formation underlies the majority of the urbanized area and exhibits moderately to highly expansive characteristics over approximately ⅓ of its outcrop area (Figures 3 and 11). Soils which form from the weathering of Pierre Shale typically exhibit some swelling. Swell pressures ranging from 3.0–6.0 GPa (3,000–6,000 psf) are not unusual. The foundation suitability of the Pierre Shale depends on building load, foundation design and change in moisture content after construction. The Pierre Shale generally exhibits a low shear strength. Much of the formation will expand moderately and exert moderate swelling pressures if the moisture content increases. Slope stability is fair to good where slopes are undisturbed or where bedding dips into the slope. Locally, slope stability can be poor for steep cuts exceeding 6 m (20 ft) in height and for cuts where bedding dips in the same direction as the hillslope but at a smaller angle relative to the slope, especially if a bentonite or clay layer is present. If these conditions are present, the Pierre Shale can exhibit progressive slope failure which migrates headward and can be difficult to arrest. Excavation is generally possible to depths of 4.6 m (15 ft) with most available construction equipment. If excavations are left open, slaking of the formation is common.

The remaining sedimentary rock formations are present only as narrow outcrop bands paralleling

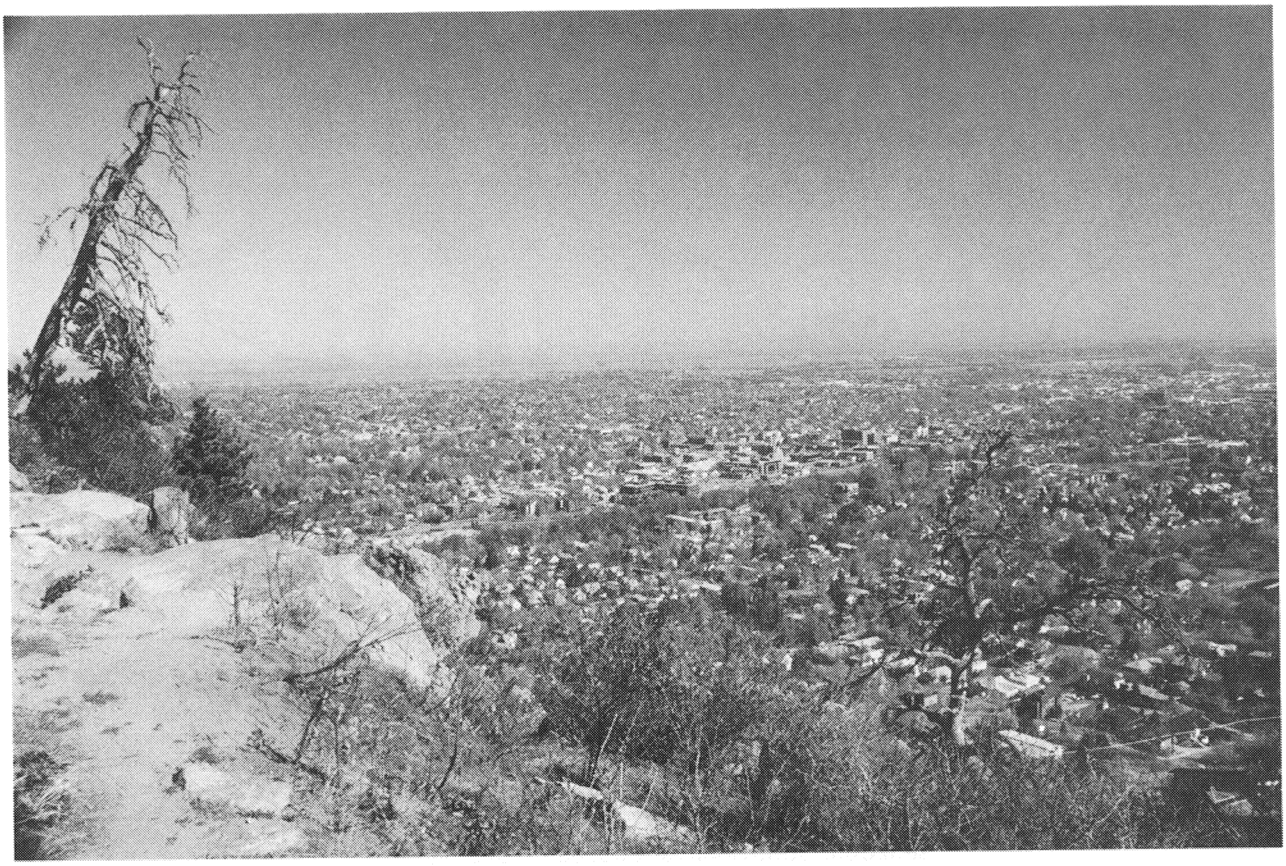


Figure 12. View of Boulder looking east from Flagstaff Mountain.

the mountain front (Figure 3). Most of these formations present few foundation problems. Due to their steeply dipping bedding attitudes, landslides and rockfalls locally present problems where the formations are undercut or where dip slope conditions exist. Some claystones and shales present in this area exhibit moderate to high swell potential.

Crystalline rocks are present in the sparsely developed, western mountainous portion of Boulder County. Few foundation problems are present in this area, with the exception of slope stability. There is a moderate risk from rockfalls and small debris slides along the steep slopes of valleys.

Exploration and Testing Methods

When conducting geotechnical exploration of specific sites within Boulder County, the first step is generally to review available technical literature. There are numerous geologic and soil reports which cover the county area, and several quadrangles have been mapped by the U.S. Geological Survey at 1:24,000 scale. These maps cover a range of topics of interest, including bedrock geology, surficial deposits, and engineering properties (Malde, 1955; Wells, 1967; Wrucke and Wilson, 1967; Gardner, 1968, 1969; and Trimble, 1975). The Soil Conservation Service has also published a soil survey of Boulder County (Moreland and Moreland, 1975).

Subsurface exploration for engineering geological purposes is accomplished through the opening of trenches or boreholes. Trenches are generally opened to a depth of 3 m (10 ft) with tire or track-mounted backhoes. Borings are drilled with continuous flight augers, hollow stem augers or rotary wash drilling rigs. Samples are obtained with the Standard Split Spoon, Shelby Tubes or California (Ring) Samplers. Standard penetration tests are commonly used to determine *in-situ* consistency of soil and weathered rock materials. Rock core drilling is used where bedrock information is needed.

The most common laboratory tests performed include: Atterberg limits, grain size distribution, moisture content and dry density, direct shear, and one dimensional consolidation-swell tests. Tests are

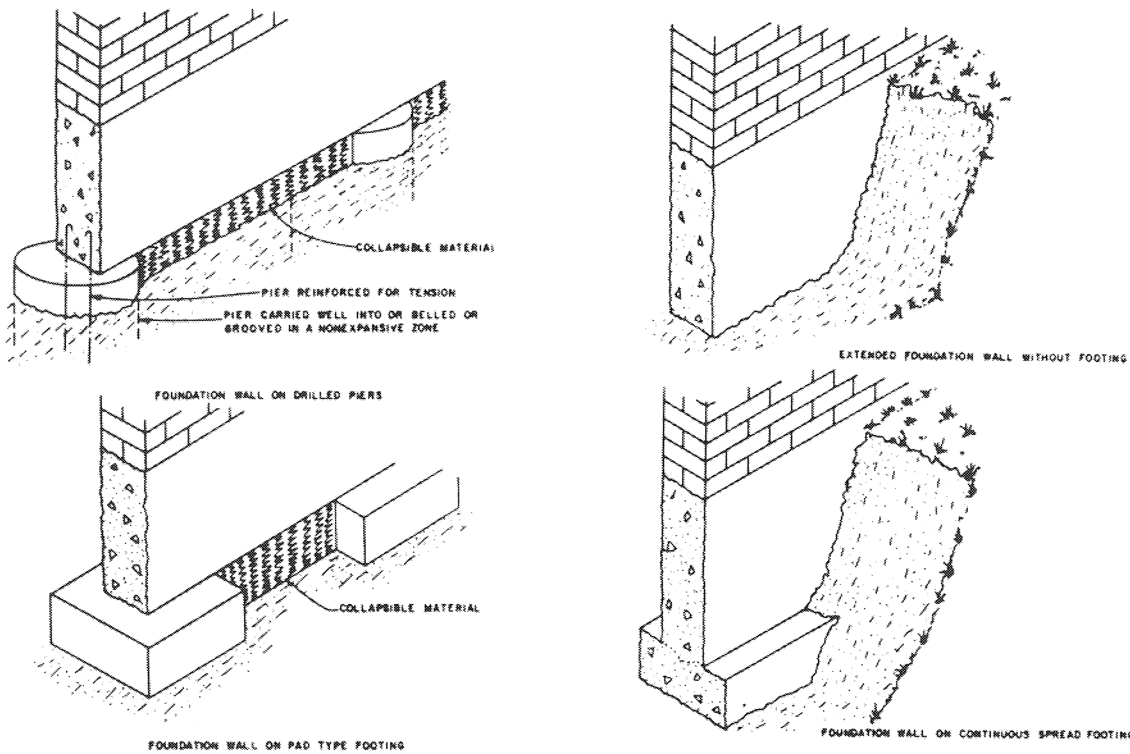


Figure 13. Typical foundations in use in Boulder County (from Holtz and Hart, 1978).

generally performed to published reproducible standards such as the American Society for Testing and Materials (ASTM) or other soil testing laboratory manuals or standards.

Typical Foundation Types in Use

The City of Boulder limits the height of buildings to 16.8 m (55 ft). This ordinance was enacted to preserve the spectacular mountain view from the downtown area as well as to maintain a small-town atmosphere (Figure 12). In Boulder County, similar ordinances restrict building heights in residential areas to 10.7 m (35 ft) and buildings in industrial and agricultural areas to 15.2 m (50 ft). These restrictions limit the variety of foundation types needed in the area. Typical foundations used are continuous spread footings, pads with grade beams, wall-on-grade, or drilled piers (caissons) with grade beams (Figure 13).

Much of the downtown area is underlain by river sand and gravel with bearing capacities of 4.0–6.0 GPa (4,000 to 6,000 psf). Most of the buildings in this area have continuous spread footing type foundations. Although most of the footings are constructed of concrete, there are some in older structures that are constructed of Lyons Sandstone slabs stacked upon each other and mortared together (Nasiatka, 1984). Where the ground-water level is high some river sands will flow into open excavations. This can undermine adjacent structural foundations. Driving steel sheet piling along the proposed excavation has proven effective. When the Exeter Building, at 1050 Walnut Street, was under construction, the adjacent building had to be underpinned to prevent foundation distress.

In areas where expansive soil has been identified, certain methods of design, construction, and maintenance have been used to avoid structural damage. Where soils have moderate to high expansion potential, drilled pier and grade beam foundations are typically used. This method of foundation design is typical of the Front Range area. The drilled piers are extended below the zone of seasonal moisture change and are designed so that foundation loads are concentrated to withstand uplift pressures from expansive soils. The grade beams are constructed so that a 7.6–15.2 cm (3–6 in.) space exists between the expansive soil and the bottom of the beam. This allows the soils adjacent to and beneath the foundation to expand without generating uplift pressures against the base of the grade beams (Figure 13). When drilling the holes for the piers, care must be taken to prevent enlargement of the top of the hole. If the top of the drill hole is enlarged and no forms are used to prevent the concrete from filling the void, the result is a mushroom-topped drilled pier. Expansive pressures have been known to break or lift mushroom-topped drilled piers out of the ground.

Where soils have moderate to low expansion potential, pads with grade beams and/or wall-on-grade foundations are often used. These foundation systems are similar to the drilled pier and grade beam foundation in that they are designed to concentrate structural loads sufficiently to resist uplift pressures resulting from soil expansion.

In most instances, continuous spread footings are generally not recommended for areas with swell potential. In contrast to the foundation systems described above, continuous spread footing foundations distribute structural loads and are therefore less suited to withstand pressures generated by soil uplift.

A segmented interior design is generally recommended in structures founded on expansive soil. Most ground or basement floor slabs constructed on expansive soils are "floating." That is, they are generally designed with a joint separating the slab from the structural or load bearing walls. If the interior non-load bearing walls are tightly attached to the slab, the upward movement can cause interior damage.

Positive drainage away from the foundation is another important aspect of building on expansive soils. Hart (1974) recommends 30 cm (12 in.) of vertical fall in 3 m (10 ft) around the building. All downspouts and splash blocks should allow roof runoff to be discharged at least 1.2 m (4 ft) away from the building. In areas of heavy lawn watering, properly directed peripheral drain pipes are effective in helping to prevent water from collecting around the foundation. Grass, shrubs, and sprinkler systems should also be kept a minimum of 1.2 m (4 ft) away from the foundation. Trees should be planted no closer than 4.6 m (15 ft) from foundations.

Construction problems related to swelling soils commonly include mushroom-topped drilled piers, lack of adequate expansion void space between soils and grade beams, allowing clays to dry excessively before pouring concrete, allowing water to pond near the foundation during construction, building without allowance for basement or ground floor movement, and improper landscaping and surface drainage (Shelton and Prouty, 1979).

ECONOMIC DEPOSITS

The Colorado Division of Mines formerly published an annual summary of mineral industry activities (Colorado Division of Mines, 1980). This practice was curtailed after 1980, hence the latest readily available information regarding production and sales of minerals in Boulder County is from this 1980 publication.

Sand and Gravel

Sand and gravel deposits of economic value in Boulder County are located primarily within the physiographic flood plains of South Boulder, Boulder, and St. Vrain creeks (Figure 14; Schwochow et al., 1974). These deposits occur within the Louviers, Broadway and Piney Creek alluviums and are the prime commercial mineral deposits in the county, in both volume and value. An inventory of these deposits shows that out of an original estimated inventory of 446,024,000 metric tonnes (439,000,000 tons), 12 percent or 54,864,000 metric tonnes (54,000,000 tons) have been extracted to date, 52 percent or 231,648,000 metric tonnes (228,000,000 tons) remain technically available for extraction, and 36 percent or 159,512,000 metric tonnes (157,000,000 tons) have been lost for extraction due to urban development (Boulder County Planning Commission, 1984).

Sand and gravel are used mainly in the production of concrete aggregate, for use on construction and paving projects. A large amount is also consumed in the production of asphaltic concrete. Because sand and gravel have a low unit value, they cannot be hauled far and remain a profitable commodity. In 1980, the dollar value of sand and gravel mined in Boulder County was over $5,500,000 (Colorado Division of Mines, 1980).

The City of Boulder and Boulder County have developed comprehensive planning programs for the extraction of sand and gravel materials. This was in direct response to the Colorado Legislature House Bill 1529, which was enacted in 1973 to prevent land uses that could prohibit extraction of valuable sand and gravel resources in the area. Because most of Boulder's high quality gravel deposits are located in flood plains, extraction pits often extend below the ground-water level. Portions of one area of extensive mining, Sawhill Ponds, in the Boulder Creek flood plain, have been reclaimed and converted into Walden Ponds Wildlife Habitat (Figure 15). Active

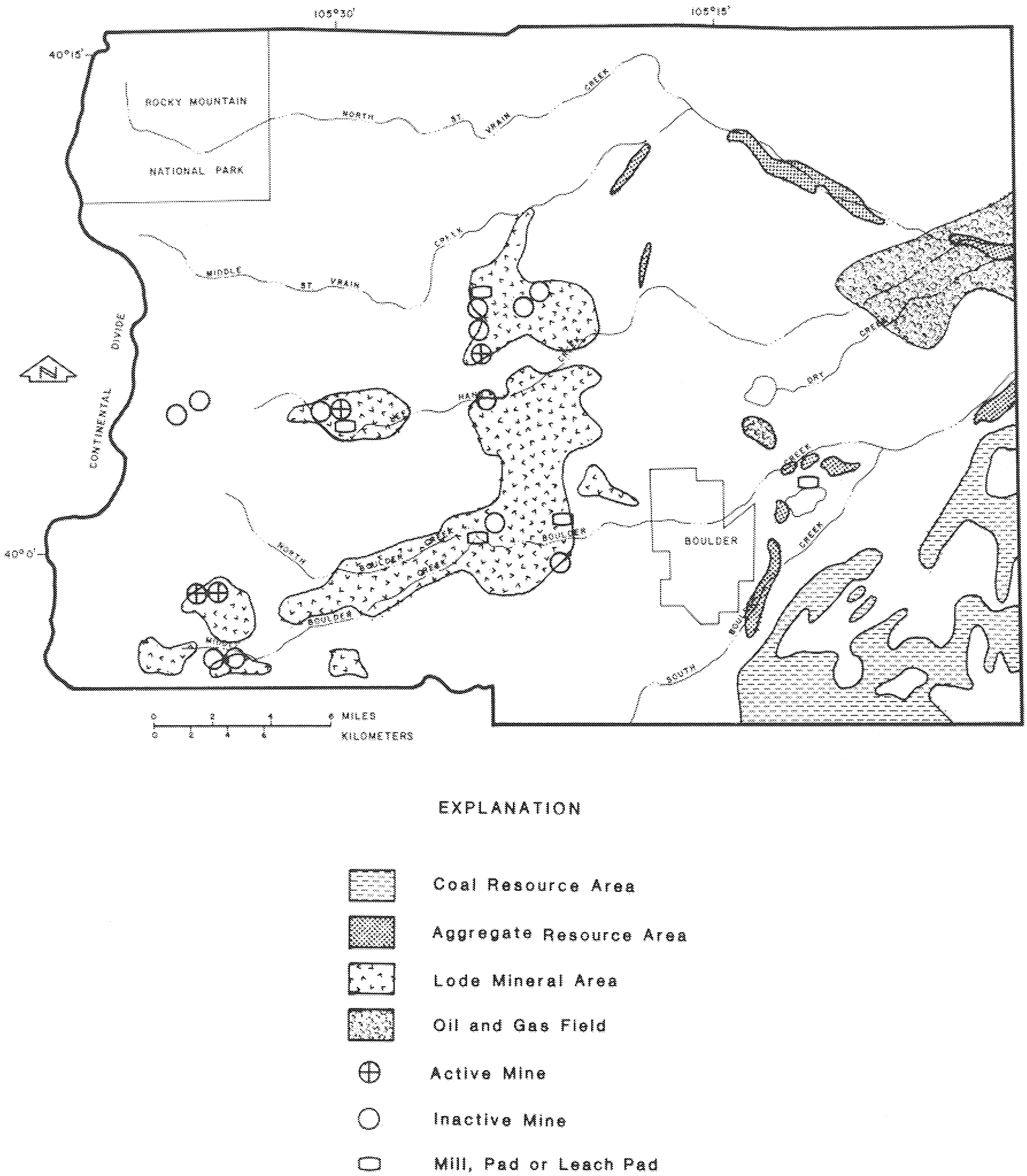


Figure 14. Map showing mineral and fuel resource areas and mining operations. Adapted from Boulder County Planning Commission (1978) and Hornbaker (1984).

mining is continuing today on the margins of these reclaimed, water-filled extraction pits.

Clay Minerals

Eastern Boulder County has some reserves of clay minerals suitable for production of brick, tile, blocks, pipe, and culverts. Good quality refractory clays are limited to the South Platte Formation of the Cretaceous Dakota Group in the south-central part of the county (Madole, 1973). Amost all clay produced in Boulder County comes from the El Dorado Springs-Superior-Marshall area (Figure 1). A limited amount of clay is mined near manufacturing sites, where it is used as an additive or supplement

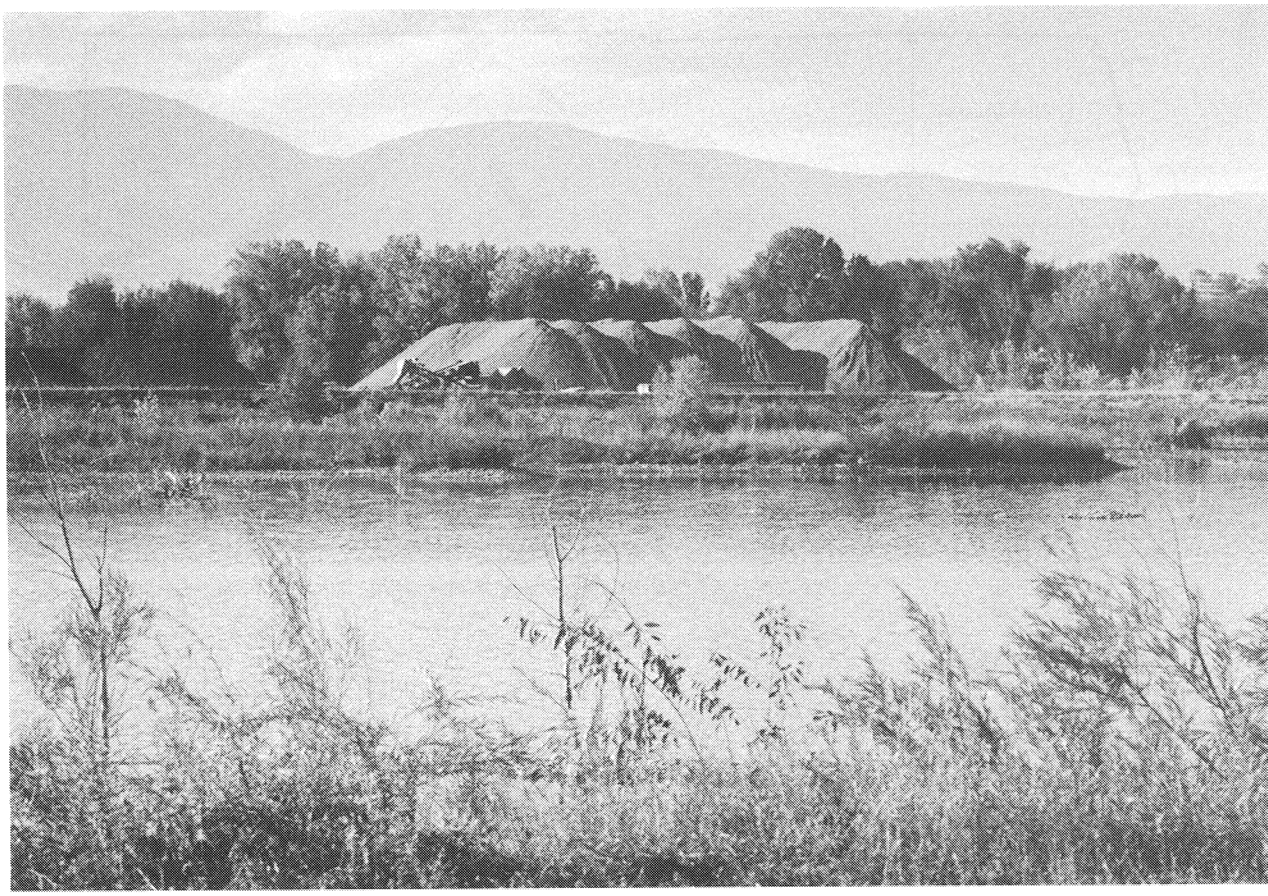


Figure 15. Sand and gravel operations adjacent to Walden Ponds Wildlife Habitat; location 9 on Figure 1.

to clay that is imported. The Lykins, Morrison, and Benton formations contain clays that can be used in this manner. In 1980, over $25,000 worth of clay was mined in the county (Colorado Division of Mines, 1980).

Limestone and Cement

Limestone is currently being mined 18 km (11 mi) north of Boulder by the Martin Marietta Corporation (Figure 1). The Fort Hayes Limestone Member of the Upper Cretaceous Niobrara Formation is the principal source. Cement is produced in just three plants in Colorado; the Martin Marietta plant north of Boulder is one of them (Figure 16). In 1980, the dollar value of cement produced in Boulder County was approximately $14,000,000 (Colorado Division of Mines, 1980).

Fluorspar

Fluorspar is mined by Allied Chemical Company in the Jamestown mining district, west of Boulder. Fluorspar is essential in the chemical, steel, aluminum, and ceramic industries. In the U.S., it is presently the only commercially developed source of the element fluorine from which hydrofluoric acid and fluoride compounds are manufactured (Madole, 1973).

Stone

Building stone in Boulder is derived almost exclusively from the Permian Lyons Sandstone near the City of Lyons, 20 km (12 mi) to the north, where the formation is well exposed and accessible to heavy equipment. Many of the older buildings around town are constructed almost entirely of this eolian "flagstone." Nearly $450,000 worth of stone was mined in Boulder County in 1980 (Colorado Division of Mines, 1980). Several old sandstone quarries are located along the mountain front in the southwestern part of Boulder.

Silica Sand

Silica sand has been mined locally in Boulder County for use in the manufacture of cement and

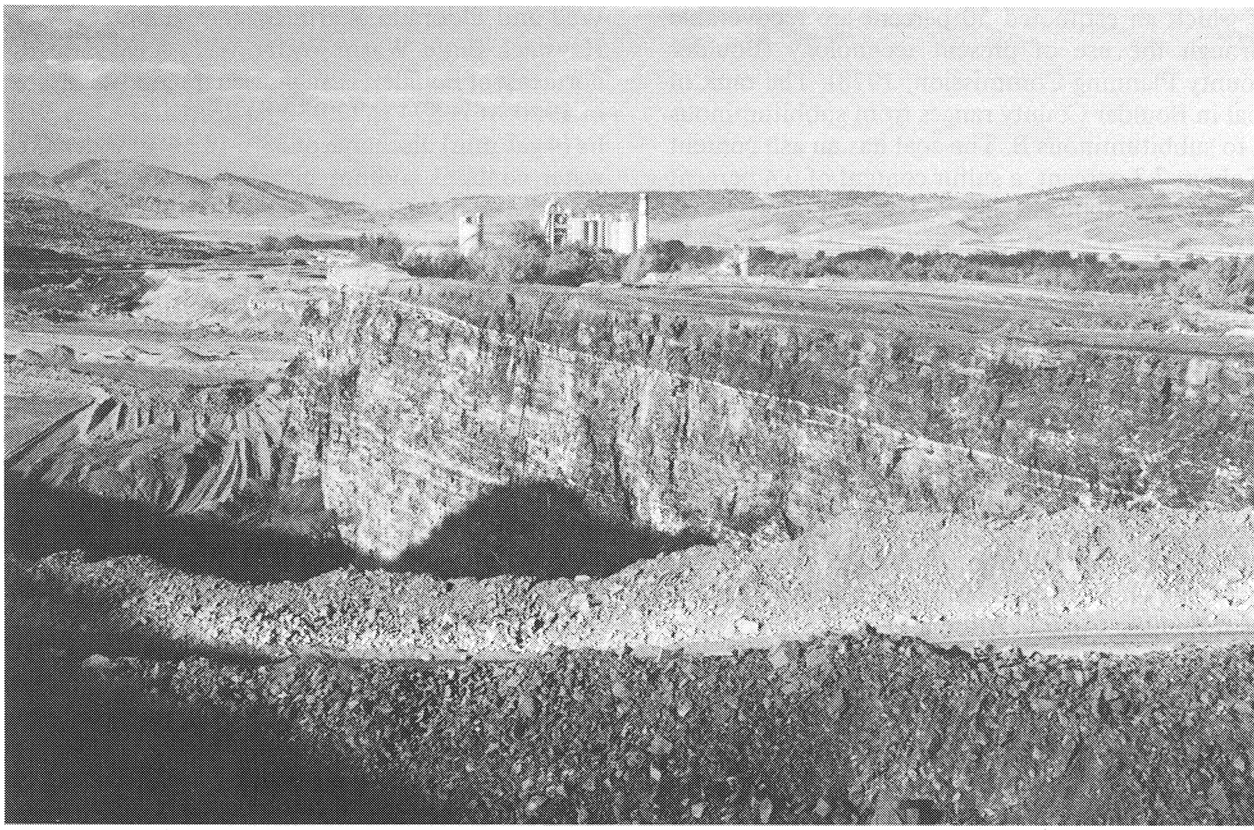


Figure 16. Martin Marietta cement plant in background, limestone quarry in Fort Hays Limestone member of the Niobrara Formation in the foreground; location 1 on Figure 1.

clay products. The principal source is sandstone in the Cretaceous Dakota Group. Martin Marietta Corporation is the major local consumer of silica sand.

Coal

Coal of economic importance occurs principally in the Boulder-Weld Coal Field southeast of Boulder (Figure 14). The Upper Cretaceous Laramie Formation is the principal coal-bearing unit. Three main coal beds were mined from the lower 38 m (125 ft) of the formation. The coal beds range in thickness from a wedge-edge to 12 m (40 ft) (Spencer, 1961). The coals were deposited in a delta plain depositional environment and are locally associated with a major northeast-trending fault zone 16.6 km (10 mi) wide and 50 km (30 mi) long, named the Boulder-Weld fault zone by Davis and Weimer (1976) (Figure 3). The horsts and grabens that were formed in the complex system of faults controlled the deposition of the coals as they are thickest in the grabens and thin over the horsts. Seismic reflection data suggest that these are aseismic synsedimentary growth faults related to the deltaic sedimentation which was, in turn, localized by Late Cretaceous movement on basement faults (Davis and Weimer, 1976).

Coal mining began as early as 1863 near Marshall. This area is one of the oldest coal mining areas in the Western United States (Kirkham and Ladwig, 1979). Most early mines consisted of adits dug into coal seams that were exposed at the surface; a few were open pit mines, with most of the remainder being underground room and pillar mines. Some areas above abandoned coal mines are experiencing subsidence (discussed in the Geologic Constraints section). Coal mining in the Boulder-Weld Field peaked in 1929. Production and mining activity has gradually decreased since that year and no coal mines are known to be active since 1976 (Kirkham and Ladwig, 1979; Bryan, 1984). Original reserves were estimated at 472,582,240 metric tonnes (465,140,000 tons). Remaining reserves are approximated at 384,393 metric tonnes (378,340 tons) of which an estimated 50 percent are recoverable through the use of present technology (Boulder County Planning Commission, 1978). The rank of coal in Boulder County ranges from subbituminous A to subbituminous B. The coal has an ash content of about 7.3 percent, a sulfur content of 0.4 percent and contains about 12,280 BTU's per pound (Boulder County Planning Commission, 1978). A study conducted in 1978 suggested that coal could not be extracted for less than 133 percent of the market price, excluding transportation costs (Pendleton, 1978).

Loam and Peat

Peat and loam have been mined commercially at Beaver Reservoir, Lefthand Reservoir, and Brainard Lake (Madole, 1973). The net dollar value of loam and peat mined from 1952 to 1971 amounted to $320,054. Peat reserves are limited to westernmost Boulder County. Although they have not been estimated officially, the reserves are considered substantial.

Oil and Gas

Petroleum has been produced in Boulder County since 1901, when the Boulder Field was discovered. Other fields have been discovered since then, although the area has never become a significant petroleum producer (Figure 14). Production from the Codell Sandstone, Hygiene Sandstone Member of the Pierre Shale, fractured zones in the Pierre Shale, and sandstones of the Dakota Group has increased since the prices of oil and gas skyrocketed in the 1970's. Discovery of the Boulder Valley Field about 11 km (6 mi) east-northeast of Boulder in 1979, which produces from the Codell Sandstone, is considered to have been the catalyst that sparked the current Codell play in the Denver Basin (Petroleum Information, 1983). Production from the Boulder Valley field began in 1981 when the wells were connected to pipeline. In December 1980, there were 10 producing wells filed within the county, accounting for nearly 7,000 Bls of oil and 70,000 Mcf of gas (Colorado Oil and Gas Conservation Commission, 1980). The dollar value of the oil and gas produced was approximately $250,000 (Colorado Division of Mines, 1980).

Geothermal Resources

In the vicinity of Boulder, two areas have been identified which possess warm water resulting from geothermal activity: the Haystack Butte Warm Water Well and Eldorado Warm Springs (Figure 1). The Haystack Butte Warm Water Well, located to the northeast of Boulder, is an unused oil test well drilled in 1920. It is 894 m (2,932 ft) deep and has a 0.25 l/s (4 gal/min) discharge of 28°C (82.4°F) water. The water contains sodium bicarbonate with approximately 1,200 mg/l total dissolved solids. Attempts to plug the well were unsuccessful, and the seeping water was used for a wading pool in the 1920's and 1930's and later as a baptismal font by a religious group. At present, the water is used in a swimming pool and to water game birds (Barrett and Pearl, 1978).

Geologically, the well is located on the crest of a southward plunging faulted anticline (old Boulder Oil Field). The heat source for the geothermal water is probably associated with the emplacement of the Tertiary igneous rocks described in previous sections of this paper (Barrett and Pearl, 1978). This anticlinal geothermal reservoir is approximately 1.4 to 4 km^2 (0.54 to 1.54 mi^2) in size.

Eldorado Warm Springs, located about 13 km (8 mi) south of Boulder, along South Boulder Creek, contains natural springs and drilled wells. The springs and wells are used to warm a swimming pool which has been the central attraction for a popular resort since the late 1800's. Water from the spring has also been bottled and sold commercially. The temperature of the warm water varies annually from 24°C to 26°C (75°F–79°F) and contains calcium sulfate. Total dissolved solids range from 84–101 mg/l (Barrett and Pearl, 1978).

Geologically, the warm water flows from springs in the South Boulder Creek alluvium, which overlies steeply dipping sandstones of the Fountain and Lyons formations. The mechanism for heating has been attributed to circulation through faults and fractures in the igneous and metamorphic basement complex to the west of Eldorado Springs (Barrett and Pearl, 1978). The geothermal reservoir extends west of the springs and covers an area of approximately 129.5 km^2 (50 mi^2).

Metallic Minerals

Metallic minerals have been mined in western Boulder County since 1859, when gold was discovered at Gold Hill. Six districts have been established: the Central (Jamestown), Gold Hill (Salina, Rowena, and Sunshine), Magnolia, Sugarloaf, Ward, and Grand Island (Cardinal, Carbon, Eldora, and Nederland) (Figure 14). The metals produced include gold, silver, tungsten, copper, lead, zinc, tin, and uranium (Madole, 1973). The dollar value of gold, silver and tungsten mined was nearly $330,000 in 1980 (Colorado Division of Mines, 1980).

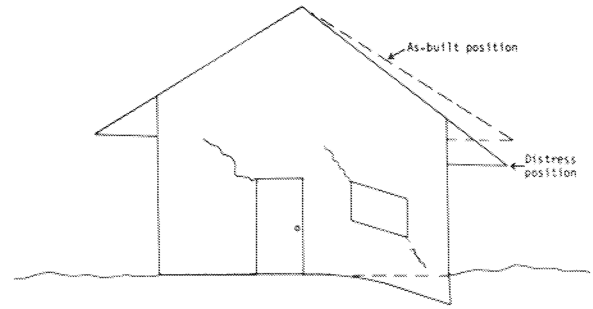
GEOLOGIC CONSTRAINTS

Expansive Soil

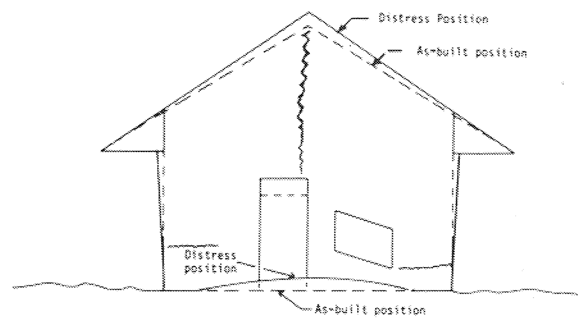
Expansive soil is one of the most prevalent and costly geologic problems in Colorado. Swelling soil damage to public facilities in Colorado costs approximately $16,000,000 annually (Shelton and Prouty, 1979). The problem with swelling soils in Colorado is so severe, in fact, that the Homeowners Warranty (HOW) Insurance Program suggested in 1980 that over 50 to 70 percent of its claims occurred in the Front Range region as a direct result of moisture sensitive soils (Lord, 1980).

In the Boulder area, swelling soil is the result of the inherent clay mineralogy. Soils derived from parts of the Laramie Formation and the Pierre Shale commonly contain montmorillonite and other smectite clay minerals. The crystal structure of these clay minerals allow them to absorb water between atomic lattice layers. As a result, these minerals increase in volume when wetted, and shrink when dried. A sample of pure montmorillonite can swell up to 2,000 percent of its original volume; however, most natural soils rarely swell more than 50 percent of their original volume (Hansen and Crosby, 1982; Hart, 1974). Expansive forces are capable of exerting pressures of 19.9 GPa (20,000 psf) or greater on confining structures such as foundations (Shelton and Prouty, 1979). Swell pressures of 4.0 GPa (4,000 psf) are very common in areas of Boulder County where montmorillonitic soils exist. The ultimate volume change of a soil sample is dependent on the percentage of smectite clay minerals present, existing moisture content, soil permeability, soil density, past strain history, extent of seasonal wetting and drying, and confining pressures (Hansen and Crosby, 1982).

Swelling soil conditions in Boulder represent the most widespread geologic constraint for development (Figure 11). Differential movement resulting from expansive soils has caused structural damage to many buildings in Boulder County (Figure 17). To reduce the potential for structural distress, common construction practice is directed towards either minimizing the potential for moisture variations which cause soil expansion or using foundation systems capable of withstanding or avoiding soil movement. According to Holtz and Hart (1978), damaging soil moisture variations can occur due to: 1) boundary effects of wetting or drying on a covered area, 2) seasonal or cyclic weather changes, 3) drainage problems, 4) landscape watering and 5) removal of soil moisture through evapotranspiration by local vegetation.



Foundation settlement (typical).



Foundation upheaval (typical).

Figure 17. Typical damage to buildings caused by expansive soils in Boulder County (from Shelton and Prouty, 1979).

Five construction techniques are generally effective in areas of expansive soils: 1) remove the soil and replace it with nonswelling material, 2) place foundation elements below the zone of seasonal moisture change, 3) install impermeable barriers to minimize soil moisture changes adjacent to and beneath the structure, 4) chemically treat the soil to inhibit swelling and 5) concentrate foundation loads sufficiently to withstand uplift pressures. Methods for testing for expansive soils and a more detailed description of typical foundations used in areas of expansive soils are described later in this paper.

Floods

Since 1864, Boulder has experienced 5 major flood events from Boulder Creek. All of the floods have occurred in May or June, when snowmelt is augmented by intense rainstorms. During May 21–23, 1876, a general storm over the Boulder Creek basin

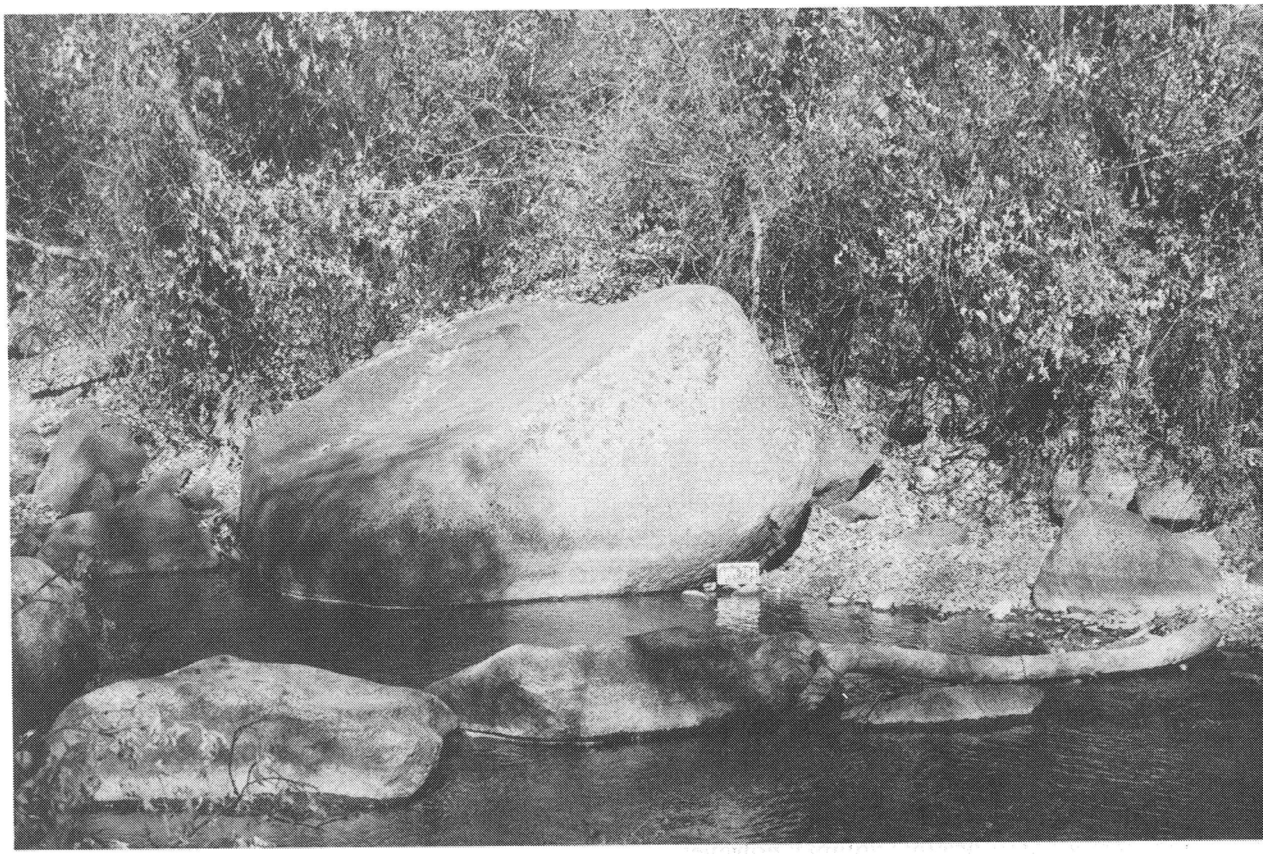


Figure 18. Large boulder in Boulder Creek located on the edge of the city; location 5 on Figure 1. Note car license plate for scale.

created flooding on the plains reportedly 2.4 km (1.5 mi) wide. Railroad service was disrupted and fences and bridges were swept away (Muller Engineering Company, Inc., 1983).

The greatest flood known in Boulder occurred on May 30, 1894, when 11.4–15.2 cm (4.5 to 6.0 in.) of precipitation fell in the Boulder Creek drainage area. Buildings, bridges, roads and railroads were washed away. Floodwaters covered the entire area from Canyon Boulevard to University Hill to depths as great as 2.4 m (8 ft). In the valley downstream from Boulder, the flood plain was reported to have been inundated to an average width of approximately 1.6 km (1 mi) for several days (Muller Engineering Company, Inc., 1983). The other major floods occurred in June 1914, June 1921, and May 1969. Evidence for the magnitude of maximum flow during floods is dramatically illustrated by the size of rounded boulders located in Boulder Creek at the mouth of Boulder Canyon (Figure 18). Analysis of the coarsest 25 percent of Boulder Creek's flood-plain alluvium near the mouth of the creek gives a mean intermediate diameter of 188 cm (6.2 ft) for these boulders. A flow of about 625 m^3/s (22,000 ft^3/s), with a velocity of 4.6 to 6.1 m/s (15 to 20 ft/s), and a depth of 3.4 to 4.9 m (11 to 16 ft) is needed to move boulders of this size in Boulder Creek (Bradley and Mears, 1980).

As early as 1969 Boulder had adopted a zoning resolution that regulated land-use practices within the 100-yr flood plain (Taylor, Alan 1984). Much of the flood planning in the foothills was reviewed after the 1976 Big Thompson flood which occurred north of Boulder. This unprecedented event was caused by a very intense rainstorm which was unusually stationary and concentrated its rainfall over one drainage basin. In 1977, the U.S. Army Corps of Engineers estimated that a 100-yr flood could result in \$22 million (in 1977 dollars) in losses in the Boulder area (Pendleton, 1978).

In 1983, Boulder County, the City of Boulder and the Colorado Water Conservation Board contracted the Muller Engineering Company to review and update the flood hazard areas for Boulder Creek. The

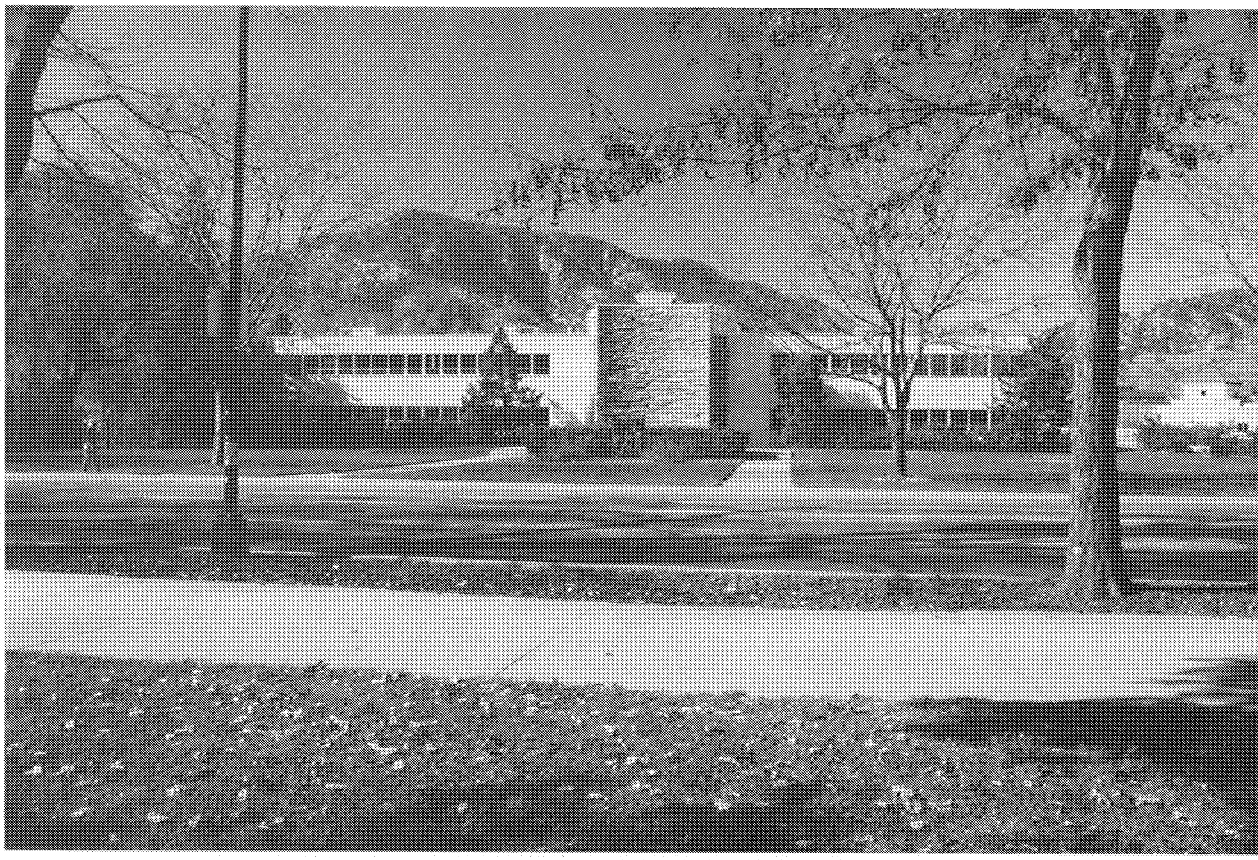


Figure 19. City of Boulder municipal building sited on the Boulder Creek flood plain less than 200 ft from the creek; location 6 on Figure 1.

resulting maps have a more accurate delimitation of the flood plain. These maps are used for planning and insurance purposes. It is interesting to note that the city's Municipal Building and Park Central Building, where the planning department resides, are located on the banks of Boulder Creek and are within the floodway of the creek (Figure 19). According to Alan Taylor (1984), the buildings were erected in the 1950's, prior to the city's flood-plain planning. The Columbia Savings building at 28th Street and Arapahoe Avenue was designed and built to withstand expected floodwaters (Figure 20). It was, however, one of the most expensive one story structures to be erected in the area.

Hydrocompaction

The major hydrocompaction problems in the Boulder area occur in soils composed of loess. Collapse-prone soils predominantly consist of silt and clay sized particles that were wind deposited in sheltered areas during glacial and interglacial periods. Typically, these deposits have a low density. Although they demonstrate high bearing strength when dry, they lose much of their strength when saturated and settle or collapse. Activities such as irrigation or lawn watering can trigger a collapse. Volume reductions are typically 10 to 15 percent (Shelton and Prouty, 1979). Surface ground displacement of up to a meter (3.3 ft) can result. Loess deposits up to 3.7 m (12 ft) thick have been discovered in downtown Boulder through subsurface exploration (Pendleton, 1978). Loess is also particularly vulnerable to wind or water erosion when stripped of its vegetative cover. Three construction techniques are generally effective in areas of collapsible soils: 1) remove the soil or place foundation elements below it, 2) prevent wetting of the soils adjacent to and beneath the structure, or 3) pre-collapse the soils prior to construction (Shelton and Prouty, 1979).

Mass Movements

Landslides, debris flows and rock falls are part of the natural erosive process. These processes are particularly active in areas of moderate to high relief

Figure 20. The Columbia Savings building, located on the flood plain of Boulder Creek, was designed with potential flood hazards in mind; location 7 on Figure 1.

which are underlain by sedimentary rocks typical of the Boulder area. Four geologic units are particularly susceptible to landsliding: the Lykins Formation, the upper part of the Dakota Group, and the weathered shale of the Laramie and Pierre formations.

The Lykins Formation underlies the strike valley between the hogbacks formed by the Lyons Sandstone and the Dakota Group. The biggest landslide problems occur on the west side of the valley where the permeable Lyons Sandstone discharges water along the dip slope into the Lykins Formation, causing many small landslides to occur.

The landslides in the upper part of the Dakota Group are found along the dip slope of the Dakota hogback. These landslides are considerably older than those present in the Lykins Formation and generally lack evidence for recent movement. These slope failures may have resulted from a time when the climate was wetter.

The Pierre Shale and the Laramie Formation provide the most significant landslide hazard to urban development because of their large outcrop area (Figure 3). Both formations contain numerous clay layers that provide low strength surfaces on which slip can occur. Landslide movement has occurred on bedding planes with less than 10° of dip when the planes daylighted in road cuts or stream channels.

Over 30 individual landslides or landslide complexes have been mapped along the foothills of Boulder County (Boulder County Planning Commission, 1984). Most of these landslides have occurred on the east slopes of the Dakota hogback and in the cut and natural slopes along the west side of the Lykins Formation strike valley. In the Marshall-Superior area southeast of Boulder, numerous additional shallow landslides have been identified on the slopes blanketed with residual soil and weathered shale of the Pierre and Laramie formations (Figure 11).

Evidence for repeated debris flow activity is present throughout the foothill stream valleys of the Boulder area. Boulders up to 5.8 m (19 ft) in diameter suspended in a poorly sorted matrix of sand and clay have been encountered near the downtown area. The extent of ancient debris flow activity extends as far as 3.2 km (2 mi) east of the foothills, covering a significant portion of the currently urbanized area. No debris flows have been reported since the settlement of Boulder in 1856 (Madole, 1973; Pendleton, 1978), but the possibility of future debris flow activity should not be ignored.

Rockfall is a hazard only in very limited areas. It occurs mainly at high altitudes along valleys with oversteepened sides and in the "narrows" of deep canyons (Madole, 1973). Rockfalls usually occur on spring days that are warm enough to thaw the surfaces of very steep slopes. They also occur during or immediately after intense rains typical of late spring and summer.

Rockfall is mainly a hazard to highway traffic, although some new housing is being built in the vicinity of rockfall susceptible areas, like portions of Boulder Canyon, South St. Vrain Canyon, the Peak-to-Peak Highway and Lee Hill Road where it parallels the Fountain Formation (Madole, 1973). The Penfold subdivision, immediately downslope from the Flagstaff Mountain parklands was threatened by the movement of a 41 metric tonne (45 ton) Fountain Formation sandstone boulder in May of 1972. The boulder had moved approximately 1 meter (3.3 ft) downslope and was precariously lodged against a Ponderosa Pine tree with a 10.2 cm (4 in.) diameter trunk. The City had the boulder broken up by hand at a cost of $6,000. Unfortunately, one 4.1 metric tonne (4.5 ton) fragment of the boulder escaped and damaged the patio and back porch of a home (Pendleton, 1978).

Avalanche hazards in the Boulder area are minimal compared to many other areas of the state. They are mainly confined to the cirques and valley-heads along the Continental Divide (Figure 11). Few well-developed avalanche chutes are present even there. The areas that become dangerous along mountain highways are shot with a cannon to release unstable snow while traffic is restricted.

Mine Subsidence and Mine Fires

Underground mining is the most important cause of subsidence in Boulder County. The magnitude of subsidence has been measured from less than a centimeter to several meters (Amuedo and Ivey, Inc., 1975; Turney and Murray-Williams, 1983). The effects of subsidence include depressions, cracks, and slumping or tilting of the ground surface. Most subsidence has occurred in the sedimentary rocks of the Boulder-Weld Coal Field in southeastern Boulder County (Figures 11 and 14). These reserves are not being developed at present, but may be mined again as future energy needs become more acute.

Many cities in Boulder County can trace their origin to mining. Louisville and Lafayette are two examples of coal mining towns in the Boulder-Weld Coal Field (Figures 1 and 14). The City of Lafayette is underlain by the Simpson Mine, which was first developed in 1888. The site was ideal because it was located only 32 km (20 mi) north of Denver and adjacent to two railroad lines. This mine was capable of an output of approximately 1,220 metric tonnes (1,200 tons) per day. The coal at the Simpson Mine lies in a horizontal plane about 73.2 m (240 ft) from the surface. The coal seam varies from 2.4–4.3 m (8–14 ft) in width, with an average of about 3.7 m (12 ft) of "clean" coal. In April 1906, *Mines and Minerals,* a mining and metallurgical journal published in Scranton, Pennsylvania described this mining operation as follows:

"The system of workings used is that known as the room and pillar method. The roof is rather soft so in driving entries and working rooms only about 9 feet of coal are taken out leaving the remaining 3 feet of coal to support the roof. When the rooms are worked out, the pillars are drawn and this 3 feet of coal is removed with them. With this system of working, little timbering is needed and the cost of production is kept at a minimum. Entries are driven in pairs 10 feet wide, the face entries being 60 feet centers and the butt entries 50 feet. The rooms are 22 feet wide and are worked to a depth of 200 feet. The room necks are 25 feet long. Pillars 18 feet wide are left between rooms and are drawn back after the rooms have been worked to their full length. Very few cross bars are required and in timbering only straight props are used."

This description is representative of the type of mining that occurred in Boulder County during the late 19th and early 20th centuries. When the local cities were founded, little thought was given to the location of structures with respect to the underground workings. Consequently, some buildings, streets and utilities have been damaged due to subsidence over the mines.

Mine fires can be started by natural or human-induced conditions. Once coal is burning in the subsurface it can burn for years or decades. Mine fires have been burning for years less than 4.8 km (3 mi) southeast of Boulder, near the town of Marshall (Figure 1). In that area, collapse of the overlying strata occurs as the fire consumes the coal. This collapse creates cracks and fissures through which oxygen is drawn in to feed the subsurface fire. Smoke and heat can also be released through the subsidence-induced fissures.

The extent, duration, and time of collapse of ground over abandoned underground mines is difficult to predict. Subsidence is generally related to the thickness of the coal removed, mining method used, the structural integrity of overlying rock strata, and the depth of mining. Subsidence has been reported at 41 localities in Boulder County (Boulder County Planning Commission, 1984). An irrigation ditch was affected by subsidence as early as 1941. A 2.4 m (8 ft) deep subsidence depression, measuring approximately 3 m (10 ft) by 6 m (20 ft), appeared in 1964 about 1.6 km (1 mi) west of Louisville (Figure 1). A 4.9 m (16 ft) deep collapse feature, measuring approximately 5.5 m (18 ft) by 7.3 m (24 ft), appeared August 28, 1971, in a mobile home park in Lafayette (Figure 1). In 1974, catastrophic subsidence occurred in Lafayette over subsurface mine workings that had been inactive for more than 50 years. In the Marshall area, where the depth of mining was shallow, collapse has been complete enough to actually discern the pattern of rooms and pillars in the abandoned mine from the surface collapse features.

In 1975, the Colorado Geologic Survey published a map of the subsidence features associated with the Boulder-Weld Coal Field. Using this map and other available mining maps, Boulder County developed a subsidence hazard map for the area. This map has been included in the Boulder County Comprehensive Plan at a scale of 1:158,400 (Boulder County Planning Commission, 1984). The Boulder County Comprehensive Plan and associated comprehensive plans for the mining towns now prohibit new construction in areas designated as having high subsidence potential. Much of the land within the cities of Lafayette and Louisville are zoned as having high subsidence potential. If a structure, for example, is torn down in Lafayette, the county and/or city law requires assurance that no damage due to coal mine related subsidence will affect replacement structures prior to granting a building permit.

An important factor to be aware of in planning any structure above an underground mine is that the surface area affected by subsidence is potentially larger in extent than the area from which the material has been extracted in the subsurface. The determination of the potential for subsidence in areas of previous mining is difficult because the necessary geologic data base is often incomplete and sometimes inaccurate. Mine records of the areas mined, pillars left intact, and air shaft locations were not formalized and, in some cases, have been lost (Ivey, 1978).

Shallow Ground Water

Some areas in Boulder County experience seasonally high ground-water conditions. Where these areas are left undeveloped few problems are encountered; in fact, some areas provide valuable wildlife habitats. When these areas are developed, the most common problem is basement or ground floor flooding. Encountering seasonal water infiltration problems in specific areas around the county encouraged the Building Department to address the problem at the design and permitting stage of development. Regulations now require that the highest ground-water level that might reasonably be expected to occur at the site be shown on the building plans if it was within 3 m (10 ft) of the pre-construction grade. In addition, if this "design ground-water level" was less than 1.1 m (3.5 ft) from the base of a structure, perimeter drains and sump pumps are required to be installed prior to permit approval. The practical result has been the omission of basements in areas of seasonally high ground water in order to keep the base of the structure more than 1.1 m (3.5 ft) from the design water table (Goodell, 1984).

SEISMIC SHAKING

Seismic activity in the Boulder area is considered to be low. According to the Uniform Building Code (1982) all of Colorado is located in Zone 1. This classification implies the following seismic risk: "minor damage; distant earthquakes may cause damage to structures with fundamental periods greater than 1.0 second; corresponding to intensities V and VI on the Modified Mercalli Intensity Scale" (Algermissen, 1969). Major historical events which fall into this category include: Hebgen Lake, Montana (1959); Kosmo, Utah (1934); Helena, Montana (1935); and Elsinore, Utah (1921). Based on the historic record, the potential for a major event (6.5 or greater on the Richter Scale), in the Boulder County area does exist but the recurrence interval ranges from 1,000 to 100,000 years.

Using UBC definitions, the levels of peak horizontal ground acceleration anticipated in the Boulder area should not exceed 0.04 g. This acceleration value represents a 90 percent probability level, indicating there is a 10 percent chance it will be exceeded within any given 50 year period (Algermissen and Perkins, 1976). This is equivalent to a mean return recurrence interval of 475 years or a risk of 0.002 earthquake events per year (Hays, 1980).

Some geologists in Colorado consider the seismic risk classifications based on the historic record of seismicity to be too low. The 110 year seismic history record may be too short to rule out the possibility of a major or moderate earthquake along the Front Range. Other studies suggest that the major faults have had little significant movement since the Laramide orogeny (Jacob and Albertus, 1985). The 1981 report by Kirkham and Rogers on the earthquake potential in Colorado suggests that Colorado should be upraded to Seismic Zone II and that a value of 0.1 g peak ground acceleration is necessary for engineering design purposes. In contrast to Zone I, Zone II classification suggests that moderate damage equivalent to Modified Mercalli Intensity VII is possible for structures with fundamental periods greater than 1.0 second (Uniform Building Code, 1982).

ENVIRONMENTAL CONCERNS

Water Supply

The first water supply for the City of Boulder came directly from Boulder Creek. This creek carried surface runoff water and meltwater from the Arapahoe glacier. A bond issue was authorized in 1874 for the first city water system. Around 1906, the city fathers purchased Silver Lake to get a dependable water supply high enough in the watershed to be relatively pristine. When the city purchased additional watershed land in Roosevelt National Forest in 1929, the purchase included Arapahoe glacier and made Boulder the only city in the U.S. to own a glacier as part of its water supply. Since then, Boulder has acquired the entire watershed, including 11 collector lakes.

Today, Boulder gets its water from three separate sources: the North Boulder Creek watershed; Barker Reservoir, which lies in the Middle Boulder Creek drainage basin; and the Big Thompson diversion complex, which brings water from the western slope of the Rocky Mountains across the Continental Divide (Figure 21; City of Boulder, 1972). The city owns all the land and water rights within the Boulder Creek watershed. This area is closed to the public, so that when consumption demands are high, very pure water can be delivered from Silver Lake to the city with only minimum treatment. A pipeline carries this water down North Boulder Creek to the Betasso Treatment Plant.

Barker Reservoir water is carried by a Public Service Company of Colorado pipeline to Kossler Lake. From Kossler Lake, the water piped across Barker Reservoir is used primarily as a supplemental source during the summer, when demand is high (City of Boulder, 1972; Wheeler, 1984).

The Big Thompson diversion complex was built by the U.S. Bureau of Reclamation in the 1930's. This project transfers water from Grand Lake on the western side of the Continental Divide, through the Alva B. Adams Tunnel, to Lake Estes on the eastern slope. The tunnel is approximately 6.4 km (4 mi) long and about 3.7 m (12 ft) in diameter. Trans-mountain water is strictly controlled and water from the Big Thompson project is managed by the Northern Colorado Water Conservancy District (NCWCD). The district manages 310,000 units per year. A unit yield can vary from a full acre foot to as low as 0.6 of an acre foot, depending on the yield established each spring by the district. Boulder bought water rights or shares of water amounting to about 21,000 units which it stores and treats at Boulder Reservoir.

The city is supported by two water treatment plants, one at Betasso Hill and the other at Boulder Reservoir. After treatment, the purified water is stored in five reservoirs along the foothills above Boulder: Devil's Thumb, Kohler, Chautaugua, Maxwell, and Gunbarrel.

Outside of the city limits and for uses outside of the NCWCD allocations, water supplies most often come from ground-water sources. Unconsolidated aquifers overlie sedimentary rock aquifers in the eastern part of the Boulder area and crystalline rock aquifers in the western part of the area. The Laramie and Fox Hills formations serve as the principal sedimentary rock aquifers in the area.

The unconsolidated aquifers include valley fill, eolian, alluvial, terraced and older alluviums and glacial deposits. They are generally less than 9 m (30 ft) thick but can be as much as 15 m (50 ft) in thickness. Unconfined ground-water conditions predominate in the unconsolidated aquifers and the regional direction of water movement is to the east (Hall et al., 1980). The flood-plain aquifers typically yield supplies of 378.5 l/min (100 gal/min) or more. The glacial and terrace aquifers typically yield supplies of 56.8 l/min (15 gal/min) or more. The valley

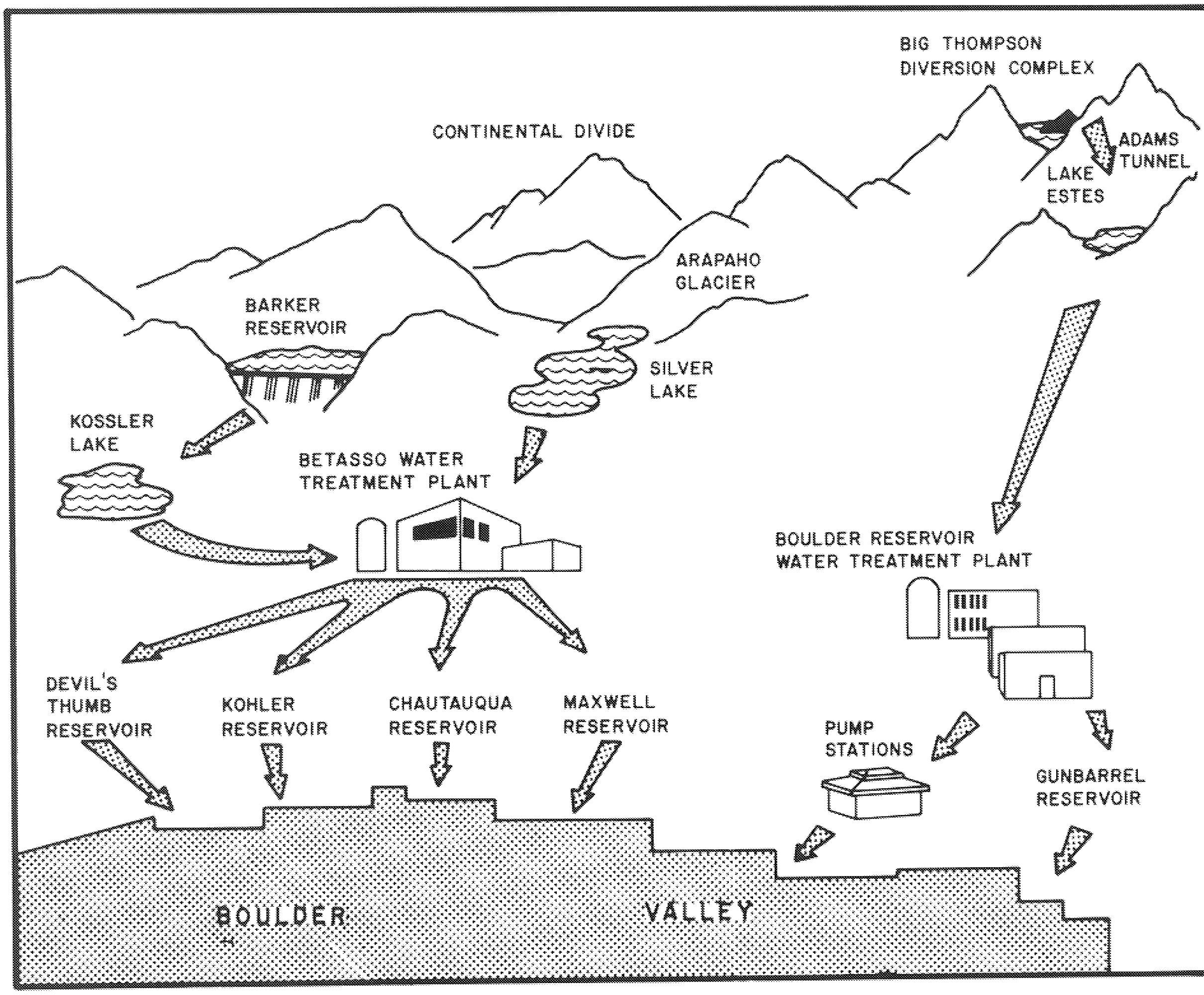


Figure 21. Boulder's water system (from City of Boulder, 1972).

fill and eolian aquifers typically yield supplies of 3.8 l/min (1 gal/min) or more (Hall et al., 1980).

Water in the flood-plain valley fill and glacial aquifers in the mountains is generally suitable for use as a drinking water supply, although bacterial contamination from septic tank leach fields can be a problem. Water in the valley fill, eolian, and terraced or older alluvium aquifers in the plains generally is not suitable for use as drinking water. Excessive concentrations of dissolved solids, sulfate, and hardness are often problems. Locally, excessive concentrations of magnesium, nitrite, nitrate, bacteria, and naturally occurring radiochemicals can be present. Water in the flood-plain aquifer is generally potable in areas just east of the mountain front. Suitability generally decreases eastward (Hall et al., 1980).

The Laramie-Fox Hills aquifer is present only in the southeastern quarter of Boulder County. This artesian (confined pressure) aquifier is about 76 m (250 ft) thick and can yield supplies of 378.5 l/min (100 gal/min) or more. The water from the southwestern portion of the aquifer is generally potable, elsewhere the water is less suitable. Excessive concentrations of dissolved solids and hardness can reduce overall water quality. Excessive concentrations of magnesium, sulfate, trace elements and bacteria are problems locally (Hall et al., 1980).

The crystalline rocks only function as aquifers where they have been fractured to an extent necessary to be permeable. They are present only in the mountains, and well yields vary significantly, depending on location and depth of the well. Water in the crystalline rock aquifers generally is suitable for use as a drinking water supply, though excessive concentrations of dissolved solids, sulfate, hardness, trace elements, bacteria, and naturally occurring radiochemicals can cause problems.

Waste Water Disposal

The City of Boulder has two separate waste water systems. One is for storm water and the other is for sewage, including domestic, commercial and industrial waste water. Storm water is collected and discharged untreated into natural drainages which feed into local streams and rivers. Sewage and industrial waste water within the city limits of Boulder is collected and piped to a waste water treatment plant at 75th and Jay Streets. The plant is capable of treating 59 million l/day (15.6 million gal/day). Hydraulically, the system can handle up to 132.5 million l/day (35 mgd), though this amount of water would not be completely treated if discharged. Normally, 37.9 to 41.6 million l/day (10 to 11 mgd) is treated (Bebler, 1983). The treatment process includes screening, grit removal, clarification, biological treatment, disinfection and, for the sludge, thickening and digestion. Sludge is disposed of through land application.

Solid Waste Disposal

The primary method of disposing of solid waste is by burial in an engineered sanitary landfill. In the Boulder urban area, solid wastes are produced primarily by urban-related activities (as opposed to industrial or agricultural-related activities). Solid waste volumes, therefore, are almost directly related to population numbers. In 1975, people generated approximately 1.9 kg (4.2 lbs) of solid waste per person per day. The estimate for 1985 is 2.3 kg (5 lbs) of solid waste per person per day (Boulder County Planning Commission, 1978). Approximately 355.6 metric tonnes (350 tons) of solid waste per day were generated in 1975. By 1985, due to population increases, it is projected that 635 metric tonnes (625 tons) of solid waste per day will be generated. There are few hazardous waste generators in the county and the volumes of hazardous wastes generated are relatively small.

Boulder County's first solid waste management plan was adopted in September 1971. This plan was developed primarily in response to the urgent need for mountain area waste collection caused by the closure of several open dumps (as opposed to modern, engineered, and permitted sanitary landfills) by the U.S. Forest Service. This plan lead to the elimination of 8 dumps in Boulder County, allowing only 2 to remain open: Marshall landfill and the Golden Rubble Landfill. The other 8 dumps were cleaned up, reclaimed, or simply abandoned. The plan also created the "green box" program, which consisted of the placement of large green dumpsters at centralized locations which served the mountain communities for nearly 10 years. Some of these "green boxes" are still in local use (Boulder County, 1982).

Today there are three certified solid waste disposal facilities that are handling waste from Boulder County. One of these facilities, Marshall Landfill, is located within the county. The other two, Longmont Municipal Landfill and Erie Landfill, are located to the northeast, in nearby Weld County.

The Marshall Landfill is the largest landfill serving Boulder County at the present time (Figure 1). Geology of the site consists of folded and faulted Laramie Formation and Fox Hills Sandstone mantled by 3–6 m (10–20 ft) of Verdos alluvium. The upper Laramie Formation is generally a poor aquifer; in contrast, the lower Laramie-Fox Hills portion of the formation is a moderate to high yield aquifer.

Prior to 1965, the area served as an illegal dump where trash and debris was simply thrown into gullies. From 1965 to 1970, a grinding and composting operation was at the site. Numerous problems resulted from this operation, including a serious problem from windblown debris. South of the original 320 acre landfill, an 80 acre site was developed into a sanitary landfill consisting of compacted soil-covered cells. According to original design plans, surface and subsurface drainage were to be controlled through contour grading and by a system of subsurface interceptor ditches. In addition, clay soil blankets were recommended to seal off all sandstones encountered. A monitoring system and a backup plan were to be provided to control the formation and migration of leachate (Boulder County, 1982). In spite of this design, leachate problems have occurred from this portion as well as older portions of the landfill (Noack, 1984). Leachates have drained into Community Ditch. This ditch occasionally adds water to the Louisville municipal water supply. The U.S. Environmental Protection Agency (EPA) has included Marshall Landfill on the National Priority List of Superfund sites. In addition to surface water contamination, ground water contamination is also suspected.

At the present time, the EPA, State Health Department, and Boulder County Health Department are attempting to determine the precise extent and specific sources of water pollution. In July of 1981, the Public Works Department developed surface drainage improvements designed to prevent direct flow of leachate into Community Ditch. As a result of these improvements, the City of Louisville was allowed to continue using the water from Community Ditch (Boulder County, 1982). A pipeline was built in the fall of 1984 to safeguard Louisville's water supply by isolating it from the leachate. Boulder County, the City of Louisville, and the landfill operator, Browning Ferris Industries, shared the cost of pipeline construction. EPA Superfund money is also available for site remediation. In the meantime, the landfill will continue to be used. It is presently estimated that the landfill has the capacity to remain open and operating until 1989.

The Golden Rubble Landfill covers approximately 16.2 ha (40 acres) within the City of Longmont. This site was used as a rubble dump prior to designation as a sanitary landfill in November 1969. The site remained active until April 1976. Approximately 95 percent of the fill material is rubble consisting of concrete, wood, and construction and demolition debris. The remaining 5 percent of the fill material consists of office wastes and household items which are non-putrescible. This landfill is not currently in use (Boulder County, 1982).

The Boulder County "green box" program was developed in 1972 to serve tourists and residents in the mountain area from Nederland to Allenspark. The original phase of the program consisted of providing 30 trash containers, each with a capacity of 4.6 m^3 (6 yds^3) at 18 selected sites along state highways and county roads for pickup by a private hauler. 44,468 m^3 (34,000 yds^3) of trash were hauled in the first 12 months of operation with noticeable reduction of roadside litter. The program received the National Association of Counties Achievement Award in 1973. In the latter part of 1980, the Board of County Commissioners (BCC) initiated the gradual phaseout of the green box program because of program costs and declining effectiveness. Some green box sites remain and the BCC has instituted user fees to cover hauling costs (Boulder County, 1982).

Collection and transportation of solid waste in the Boulder area is accomplished primarily by private contract haulers. The city of Longmont provides a municipal collection service for residential solid waste and the City of Boulder provides hauling of spring cleaning debris each year for city residents (Boulder County, 1982).

Beginning in 1983, Boulder County began conducting a county wide search for additional landfill sites. Geologic and hydrologic compatibility are high priority issues that the county is considering through the selection process. At this time, final site selection has not been made.

Wetlands Factors

Wetlands are defined as land where an excess of water is the dominant factor determining the nature of soil development and the types of plant and animal communities living at the soil surface. Thus wetlands in Boulder County include marshes, swamps, bogs, wet meadows, pot holes, sloughs, river-overflow lands, reservoirs, lakes, and streams. Wetlands are valuable to waterfowl breeding, wintering and migration habits. They can also store ground water, stabilize runoff, retain surface water, and reduce erosion. Wetlands were inventoried for Boulder County in 1977 prior to the development of the County's Master Plan. Protection of these wetlands is not guaranteed through the Master Plan, although it is generally the policy of the county to preserve these areas whenever possible (Boulder County Planning Commission, 1978).

MAJOR ENGINEERING STRUCTURES

Due to the height limitation of buildings in the City of Boulder, there are few structures which are more than 7 stories high. A notable exception to the height ordinance are the dormitories associated with the University of Colorado, Williams Village complex (Figure 22). These four dormitories range in height from 11 to 15 stories. The complex was exempt from the city and county height restriction because it was constructed on state land. Each dormitory has a basement and is founded on drilled piers approximately 1.2 m (4 ft) in diameter and approximately 15.5 m (50 ft) long.

USE OF UNDERGROUND SPACE

The use of underground space in Boulder has been primarily limited to basements and below grade parking. No other plans for the use of underground space are currently under consideration by city or county planners.

ENGINEERING GEOLOGIC PRACTICE IN BOULDER

Legislation

In 1972 the Colorado General Assembly enacted Senate Bill 35, which requires an investigation of the geologic factors that would impact any proposed

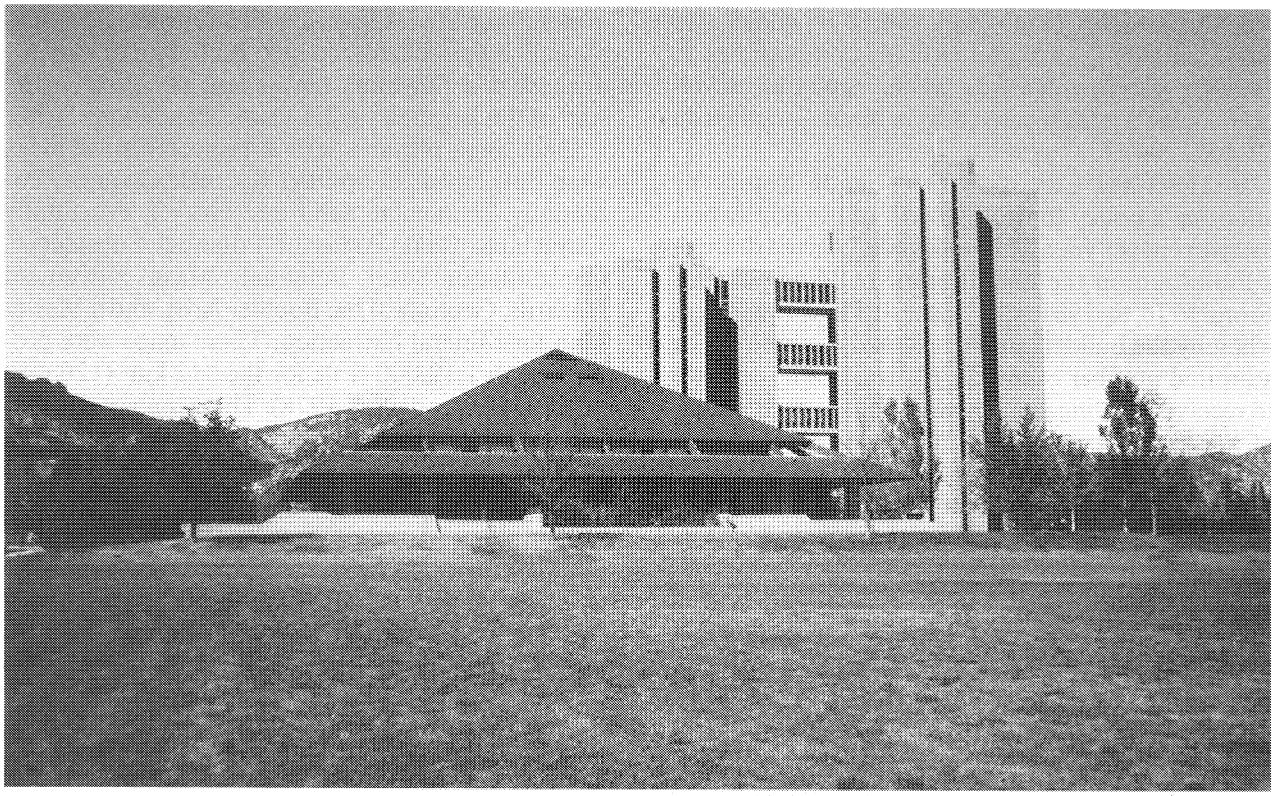


Figure 22. Williams Village dormitory towers, University of Colorado.

new subdivisions in unincorporated areas of the state. Since that time, most geologic reports have been prepared by geotechnical consultants for subdivisions and/or land developers. These reports are submitted to county planning departments, which usually submit them to the Colorado Geological Survey (CGS) for review and comment. In July 1983, the CGS began to charge $160 to $275 for this service. Approximately four CGS geologists spend a major part of their time conducting these reviews and are recognized as specialists in the areas of the state that they evaluate (Rogers, 1984). Approval or disapproval of a subdivision rests solely with the county, however, and the Colorado Geological Survey has no regulatory authority over their decision.

In 1973, House Bills 1529 and 1065 were enacted. House Bill 1529 precluded any governmental agency from zoning for exclusive use any area of mineral deposits, including sand and gravel deposits, deemed to be of economic or strategic importance. The bill also required the local government to develop a master plan for extraction of the deposits. The thrust of this bill was directed toward extraction of aggregate resources in the urbanizing areas of the state prior to development. House Bill 1065 established the Colorado Mined Land Reclamation Board to ensure proper reclamation of mined-out areas in the state.

In 1974, House Bill 1574 was passed, requiring that all geologic reports prepared for government review be done by a "professional geologist." A "professional geologist" was statutorily defined as an individual with at least 30 semester hours of undergraduate geological education and five additional years of experience which could include no more than two years of graduate work. In 1973 and again in 1976 attempts were made to enact a geologist-registration law in Colorado, but both failed.

House Bill 1041, requiring the Colorado Geologic Survey to assist local governments in identifying and designating geologically hazardous areas, also passed in 1974. Areas subject to avalanches, landslides, rockfalls, mud and debris flows, unstable slopes, seismicity, radioactivity, subsidence, expansive rock and soil, and mineral resource areas were included under the law. The law legally defined geologic hazards and authorized cities and counties to manage activities in geologic-hazard and mineral resource areas. A related bill, HB 1034, which also passed in 1974, empowered cities and counties to consider geologic hazards when regulating development and other activities in their jurisdiction (Rold, 1984).

In 1976, the City of Boulder made history by adopting a policy to limit growth of the population to 2 percent per year. This policy was enacted through a limitation on the allocation of building permits. From 1976 to 1981, the "Danish Plan" was used, whereby the builders in the area had to compete for a limited number of building permits. The projects to receive building permits were chosen on the basis of merit and location; that is, were they designed according to the best interests of the city and whether they were close to existing city services. From 1981 to 1984, the "Residential Growth Management System" was used. With this system the builders got preferential consideration if they designed moderate income housing or energy efficient housing. Phasing, or how close the project was to completion, was also a criterion in selection. Presently, the city is revising the Residential Growth Management System, although it is not expected to change substantially (Pollock, 1984). There have been numerous legal and moral challenges to the growth limitation policy, but so far the city is steadfast in using building permit allocation to follow the electorate's wishes to limit growth to 2 percent per year.

City Geologist

In 1967, the City of Boulder requested assistance from the U.S. Geological Survey (USGS) to evaluate a foothills subdivision proposal. The USGS persuaded the city to hire a geology graduate student at the Colorado School of Mines, on a part-time basis, to map open utility trenches and to compile a geotechnical map of the city. The Utility Department was especially receptive to any geotechnical input that would reduce their capital losses resulting from landslides, swelling soils, or ground subsidence. From 1969 to 1971, this staff geologist position was funded from the Utility Department's general overhead expenses. From 1971 on, the position was funded partially by direct charges to projects for geotechnical design services. In 1974, the city funded an "Urban Geology" program to analyze all geotechnically related engineering failures, to analyze and inventory all geologic hazards, to address resource conservation, and to compile single purpose derivative geotechnical maps of the urban area, among other things. By 1975, the job title "Geologist" was officially created and the job was indicated as a full-time, permanent position on the staff of the Engineering Division (Pendleton, 1978).

Eight single purpose derivative geotechnical maps were developed: Simplified Bedrock Geology, Potentially Extractable Sand and Gravel, Potentially Extractable Coal, Areas of Potential Subsidence, Consolidation/Swell Potential, Mass Movement Hazards, Geology of the Boulder Area, and a Master Plan for Mineral Extraction. These maps were produced at a 1:12,000 scale for the 312 km^2 (120 mi^2) urban area (Pendleton, 1978). These maps were used extensively when the comprehensive plan for the city was developed. An effort was made to place geologically hazardous areas in open space or to restrict them to other nonurban uses. Critical mineral resources were similarly protected (Rose, 1984).

In December 1979, the City Geologist resigned and he has not been replaced. The position was eliminated in a budgetary cut. Presently, the maps developed by the City Geologist are available only for reference and are not used in a formalized manner.

County Geologist

When the Colorado Legislature approved House Bill 1041, the General Assembly also appropriated funds to subsidize counties in application of the law (Pendleton, 1978). Boulder County used these funds (approximately $25,000) to hire a staff geologist in November 1975. He developed two maps: the Mineral Resource Areas map and the Geologic Hazard and Constraint Areas map for inclusion in the 1978 Comprehensive Plan. These maps were not traditional geologic maps but, rather, maps designed for use in long range planning by people not necessarily familiar with geology. The Mineral Resource Area map consists of general divisions which include coal resource areas, aggregate resource areas, and lode mineral areas (Figure 14). The Geologic Hazard and Constraint Area map consists of four relative geotechnical ratings of the entire county, ranging from major-extensive problems and high risk to minor-few problems and nominal risk (Figure 23). In addition to the four rankings, the map contains symbols which indicate the specific geologic hazard or constraint present, such as "l" for landslide or "x" for expansive soil. These maps are available through the county.

In conjunction with the Building Department, the

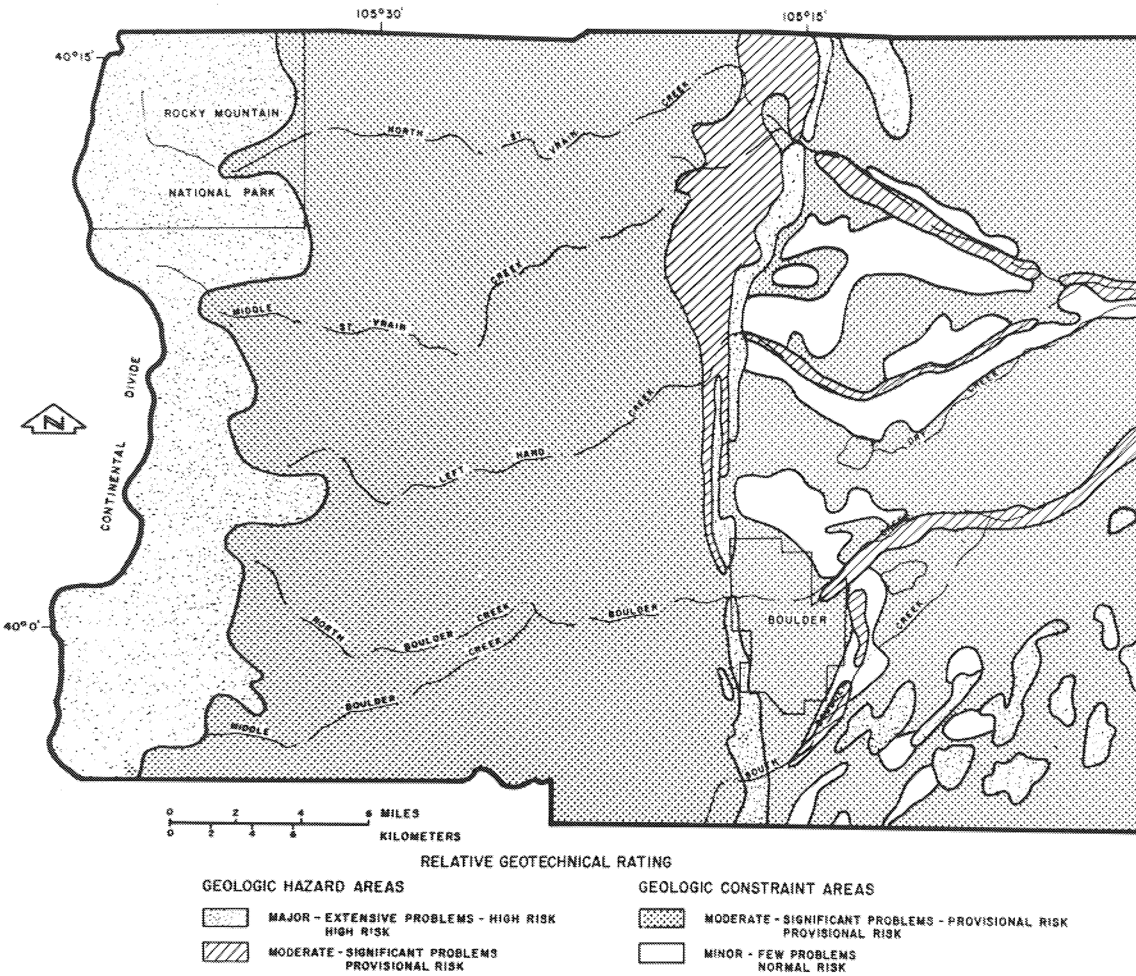


Figure 23. Map of geologic hazards and constraint areas as developed for planning purposes. Modified from Boulder County Planning Commission (1978).

County Geologist assisted in making some revisions to the building code used in Boulder County. In particular, geologic reports are usually required when the building site is located within the major or moderate geologic hazard area of the planning map or within a hillside area (i.e., 5:1, horizontal/vertical slope). In addition, where expansive soil is suspected, foundation investigations and specific soil testing procedures are required. Finally, a design ground-water level was defined as the highest ground-water level that might reasonably be expected to occur at a given site (Goodell, 1984).

The County Geologist left in January of 1980. Since then, the county planning and building officials have used the major and moderate hazard areas on the Geologic Hazard and Constraint Areas map to determine when to require a geologic report. They usually submit the report to the CGS for review and act in accordance with CGS recommendations. At present, the county has no plans to replace the County Geologist (Goodell, 1984; Bryan, 1984).

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GEOLOGY OF COLORADO SPRINGS, COLORADO

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ABSTRACT

The geology of Colorado Springs is a microcosm of the geologic complexity exhibited throughout Colorado including scenic beauty, mineral extraction history, and numerous geologic hazards. Colorado Springs is nestled in the foothills of the Front Range of the Rocky Mountains at the foot of Pikes Peak. Colorado Springs is located on the southern edge of the Denver Basin, and along the northern edge of the Canyon City embayment at the confluence of Fountain and Manitou Creeks. Rocks and deposits ranging in age from Early Proterozoic biotite gneiss to recent alluvium are exposed within the city limits and have influenced the urban development of Colorado Springs. The land within the city limits has experienced earthquakes, glaciation, catastrophic rockfall events, debris flows, and large landslides within the recent geologic past. All of these events have contributed to shaping the scenic beauty including world famous Pikes Peak and the Garden of the Gods.

The City of Colorado Springs was founded as a farming and ranching community in 1871 just five years before Colorado formally became a state. It later was a smelting center and a jumping off spot for gold seekers in the 1890's headed to the gold mines at Cripple Creek and Victor. Coal mines were active along the north and northeast sides of the city from the late 1800's until the late 1950's. The mining heritage of the city continues to this day with mining of silica sand, aggregates and limestone.

Colorado Springs has experienced numerous natural and geologic related disasters including major floods in 1921, 1935 and 1965; debris flows in 1921, the late 1950's and 1965; mine subsidence, rockfall, new and reactivated landslides in 1995, 1997, and 1999; and damage to structures and pavements from expansive soils and steeply-dipping expansive bedrock. Other

local geologic hazards include collapsible soils, elevated low-energy gamma radiation, indoor radon gas, and earthquakes.

The early growth of Colorado Springs and neighboring Manitou Springs was concentrated along Fountain and Manitou Creeks. The city generally grew between the bluffs to the north and east and the steep slopes of the mountains to the west. In the 1960's and 1970's the city began to grow past these natural barriers and onto the rolling hills to the north and east and onto the slopes at the foot of the mountains to the west. New problems related to geology were encountered during the city's growth beyond the creek valleys and onto the steeper portions of the valleys. These problems were primarily related to slope instability and were ultimately addressed by the City of Colorado Springs through a geologic hazards ordinance in 1996 that now requires a geologic hazard evaluation for most construction within the city limits.

INTRODUCTION

Colorado Springs is located about 62 mi (100 km) south of Denver at the southern end of the Rampart Range (Figure 1). The population of Colorado Springs in 2000 was approximately 361,000 residents with about 517,000 residents in the metropolitan area. Since Colorado Springs serves as the hub of activity for the Pikes Peak area, this paper includes significant geologic considerations of the greater Colorado Springs metropolitan area, which includes suburban and exurban Manitou Springs, City of Fountain, Security, Widefield, Monument and the Falcon and Colorado Center areas.

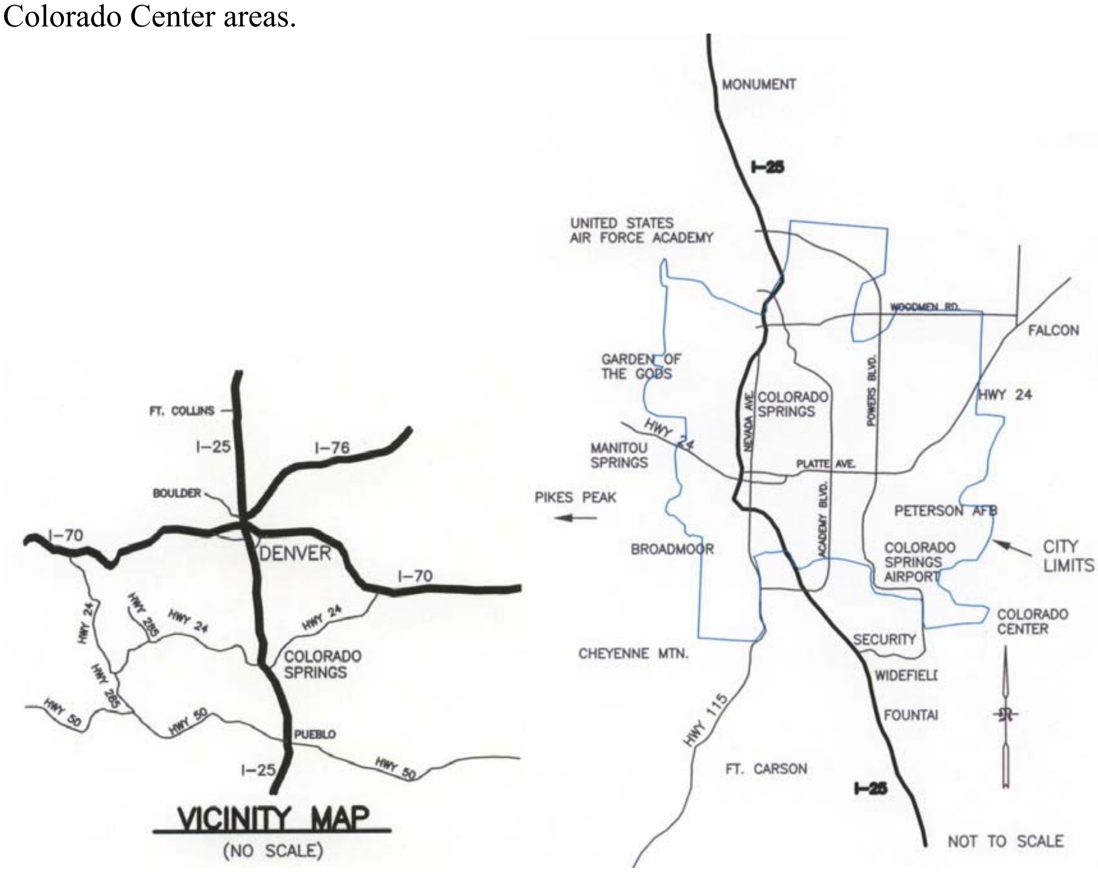


Figure 1. Colorado Springs area and vicinity maps.

Colorado Springs is situated at the foot of Pikes Peak and Cheyenne Mountain, at the confluence of Monument and Fountain Creeks, and includes the world famous Garden of the Gods. Elevations within Colorado Springs city limits range from about 5720 to 9212 ft (1743 to 2808 m), a difference of 3492 ft (1065m) (White and Wait, in press). Just a few miles west of Colorado Springs, Pikes Peak tops out at 14,110 ft (4300 m).

The largest employers in Colorado Springs include high-tech companies such-as Intel, Atmel, HP, Oracle, SCI Systems Inc., Agilent Technologies, and MCI; Memorial Hospital, Penrose-St. Francis Health Services, Focus on the Family, USAA, School Districts, local governments, Colorado Springs Utilities, and the U.S. armed forces (The Colorado Springs Business Journal, 2003). The military bases in the Colorado Springs area include: Fort Carson, Cheyenne Mountain Air Station, Peterson Air Force Base, Schriever Air Force Base (formerly Falcon Air Force Base), and the U.S. Air Force Academy. Colorado Springs is also home to the US Olympic Training Center (formerly Ent Air Force Base), the world famous Broadmoor Hotel, and Pikes Peak International Speedway.

LOCAL HISTORY

The history of the Colorado Springs area has always been tied to its close proximity to mineral and geologic resources, and its scenic geographic location. The history of Colorado Springs is actually a tale of two cities: Colorado City and Colorado Springs. Colorado City was located about 1 mi (1.6 km) west of Colorado Springs and 1.5 mi (2.2 km) east of Manitou Springs along the banks of Fountain Creek.

Colorado City was founded in 1859 as a supply town on the old Colorado Trail (Ute Pass today) serving prospectors headed to the South Park gold fields (Aldridge, 1996). It was named for the red rock formations that existed north of town, 'Colorado' meaning 'red' in Spanish. These rock formations were previously used as camps for Indians and prospectors. While surveying the areas around Colorado City, two of the town builders, M.S. Beach and Rufus Cable came across the rock formations. Beach had mentioned it was a good place for a Beer Garden. Cable exclaimed "Beer Garden! Why it is a fit place for the gods to assemble! We will call it Garden of the Gods" (Sprague, 1987). It was used as a park area near Old Colorado City, and even though it changed hands until it was bought by Charles Perkins and given to the city as a park, it was always a place where the public came to picnic, hike and enjoy the natural splendor.

Colorado City initially prospered as a supply town, being second in size only to Denver in the Colorado Territory. It was first considered as the site for the state capital (Aldridge, 1996; Hall, 1889). In 1876 when Colorado became a state, however, Denver was chosen as the new capital. A better road was built from Denver to the gold fields in South Park, and travel through Colorado City and over Ute Pass slowed. Due to lack of trade and traffic, Colorado City residents turned to agriculture and ranching to make ends meet.

General William Jackson Palmer founded Colorado Springs in 1871 (Ormes and Ormes, 1933; Sprague, 1987). He had intended to build a resort town where the wealthy could come and enjoy the healthful climate, natural scenery of Pikes Peak, Garden of the Gods, and the Soda Springs at the base of Ute Pass. Palmer bought the town site at the confluence of Fountain and Monument Creeks and also the Soda Springs in Manitou Springs at the base of Ute Pass. He brought the railroad from Denver along the Denver and Rio Grande narrow gauge railroad. Spas were built in Manitou Springs and Hotels and Sanitariums were built in Colorado Springs. Colorado Springs became a resort town for the wealthy, tourists and health seekers searching for a cure for tuberculosis. The natural scenery of the area, clear air, and climate attracted many. In fact, the words to "America the Beautiful" were penned by Katherine Lee Bates as she stood on the top of Pikes Peak in 1893 and looked out across the plains below (Sprague, 1987). Colorado Springs was called "Little London" because it had an aristocratic air with its English influence and ban on alcohol (Aldridge, 1996).

Meanwhile, Colorado City benefited from Colorado Springs' prosperity. Numerous resources were found around Colorado City including clay for making bricks, gypsum, sand and gravel aggregate, silica sand for glass making, and limestone and sandstone for building materials (Aldridge, 1996; Sprague, 1987). After gold was discovered in Cripple Creek in 1890, Colorado City once again assumed its role as supplier to the gold industry. Railroads were run from Cripple Creek to Colorado City to ease transport of ore for processing where water and other resources were more plentiful. The Short Line ran along what is now Gold Camp Road and the Colorado Midland Railroad ran up Ute Pass. Reduction mills included processes involving bromide or chlorination in the beginning, but when the Golden Cycle Mill was rebuilt in 1908 after a fire, it used the newest and most efficient cyanide processes for ore reduction and soon drove the other mills out of business (Sprague, 1987).

Coal was discovered in the northern portion of the Colorado Springs area and was mined from the Laramie Formation between 1883 and 1965 (City of Colorado Springs Planning Department, 1967). The coal was used for domestic purposes, railroads, and reduction mills in Colorado City and Cripple Creek (Sprague, 1987). Gold production in Cripple Creek fell after 1905 and the railroads that ran to Cripple Creek were eventually abandoned.

While Colorado Springs was primarily a resort town, Colorado City maintained its identity as an industrial area (Aldridge, 1996). It became known as "Old Town" or the "Westside of Little London". Since liquor was banned in Colorado Springs, Colorado City supplied the need with its saloons and red light district, and was also called "Sin City". After a time the two towns grew together and in 1917, Colorado City was annexed into Colorado Springs.

Another influence in Colorado Springs history was the decision to put a military base near the foot of Cheyenne Mountain. In 1942, Camp Carson (Fort Carson today) became post for the 89th Division and training of soldiers (Sprague, 1987). This brought more explosive growth and more military bases such as Peterson Field in 1948, Ent Air Force Base (now the Olympic Training Center), The US Air Force Academy in 1954 and the North American Air Defense Command and Combat Operation Center (NORAD) in the 1950's.

While the gold and coal industries no longer operate in Colorado Springs, the military influence, tourism, and recently religious headquarters and the computer industry continue to provide an economic base for increasing growth and development. As the city has grown, it has expanded over the bluffs that once contained it, out over the plains to the east and up the foothills to the west. This development pressure has created problems related to geology as growth pushed into areas with more severe geologic hazards.

GEOLOGIC SETTING

The mountainous peaks and rolling plains topography of Colorado Springs is the result of several episodes of uplift and erosion over geologic time. Colorado Springs sits on the edge between the Denver Basin and the Front Range. The Denver Basin is a sedimentary structural basin of Mesozoic and Cenozoic strata formed by Laramide tectonics in the early Cenozoic. It is partly concealed by middle to late Cenozoic sediments. The Front Range is considered the largest uplift in the southern Rocky Mountain Province, the core of a large late Cretaceous-early Tertiary anticline that was tilted at least twice in the middle and late Cenozoic as part of regional deformation (Steven and others, 1997). Significant uplifting and fault movements also occurred in the middle and latest Miocene, Pliocene, and as late as the Quaternary. While surface relief from Pikes Peak to Colorado Springs is over 8,000 ft (2,440 m), the maximum basement relief from Pikes Peak to Precambrian rocks below the Denver Basin is over 21,000 ft (6,400 m). Plate 1 is a geologic map of the Colorado Springs area (modified from Trimble and Machette, 1979), and Figure 2 is a bedrock stratigraphic column for the Colorado Springs area.

Geology of Colorado Springs - Plate 1.pdf

Plate 1. Geologic Map of Colorado Springs. Modified from Trimble and Machette, 1979. Note: This map is provided by the U.S. Geological Survey for public use. The Colorado Geological Survey provided digital scanned images of this map for this publication.

Precambrian

Beginning with the oldest Proterozoic rocks exposed in the mountains west of Colorado Springs, about 1.8 billion years ago (bya) thick volcanic and sedimentary strata accumulated between a chain of oceanic islands lying offshore from an continent called the Archean Wyoming Province (Reed, 2000). Plate tectonic forces lowered these ancient sediments to depths of 8 to 10 miles (13 to 16 km) below the surface. In contact with hot magma, this material fed intrusive necks, which crystallized at depth to form granitic intrusive rocks. Heat and pressure of deep burial and igneous intrusion metamorphosed the original sedimentary and volcanic rock into schists and gneisses. These rocks of sedimentary origin and hornblende-felsic gneisses of volcanic origin are exposed in Williams Canyon near Manitou Springs. Ancient metamorphic rocks were injected by molten magna at 1.71 bya (foliated biotite-muscovite granodiorite) and at 1.04 bya (Pikes Peak Granite). Erosion reduced the Colorado Springs area eventually to a low level with smooth topography (Sonnenberg and Bolyard, 1997).

Paleozoic

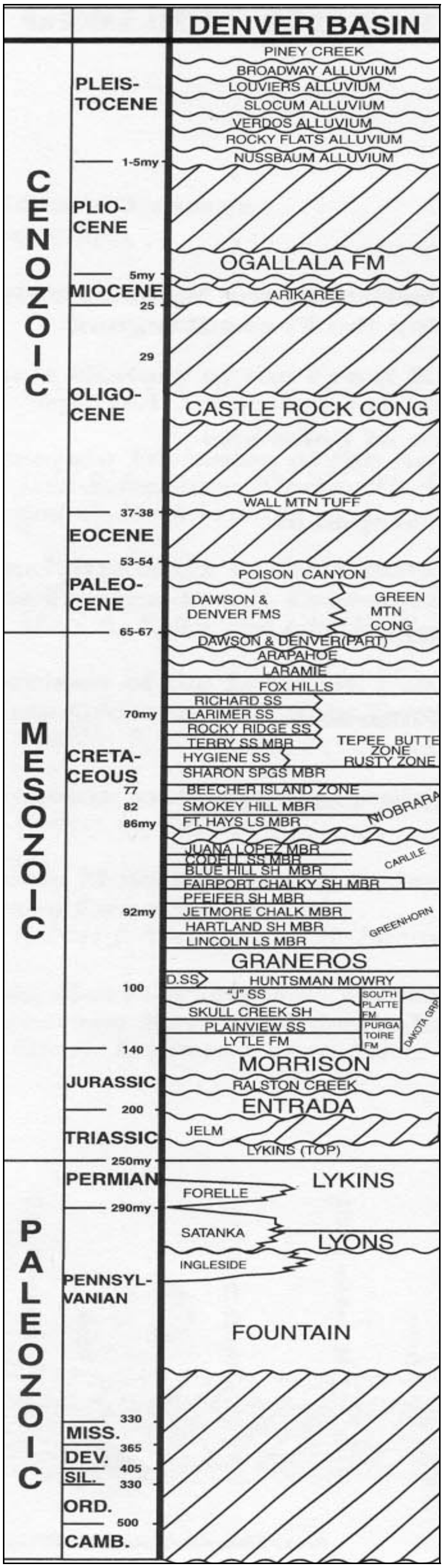
About 525 million years ago (mya) shallow tropical seas began flooding eastward across this landscape, inundating the continent. For nearly 200 million years from Cambrian through Mississippian time sand and mud sweeping into the seaway built up layers of sandstone, shale, and limestone deposits up to 1,000 ft (305 m) thick. These continental-shelf sediments were derived from the northern Transcontinental Arch and deposited and preserved near the present-day Colorado Springs in a trough known as the Colorado Sag (Harms, 1965). The Sawatch Sandstone and the Peerless Dolomite were deposited in the Lower and Middle Ordovician, respectively; and the Leadville and Williams Canyon Limestones were deposited in the Mississippian.

In Mississippian time Colorado Springs was inundated by a shallow transgressive sea from the southeast (DeVoto, 1980). The Williams Canyon Limestone represents an intertidal environment, and Colorado Springs was near a shoreline between the sea and fault-exposed older rocks of the ancestral Front Range. Paleozoic rocks exposed near Manitou Springs today (450-ft thick [137 m]) represent a small area of time not deposited in other parts of Colorado. Long periods of time are missing due to unconformities between these units, such as the unconformity between the Mississippian Williams Canyon Limestone and the Ordovician Harding Sandstone in which more than 100 million years of time are unrecorded.

During the Pennsylvanian Period the Ancestral Front Range Highland uplifted rapidly. Plate tectonic movements along a continent-continent boundary between North and South America occurred 320 mya. The uplift, which lasted about 70 million years, reactivated Precambrian basement faults into large northwest-trending structures. Enormous block-faulted mountains formed as orogenic forces raised and faulted the province west of Colorado Springs. The eroded remnants of these rocks along the Front Range are marked by the thick (>1,000 ft [333 m]) coarse-clastic Pennsylvanian-age Fountain Formation.

The red and white conglomeratic and arkosic rocks of the Fountain Formation were shed from a source to the west, on the hanging wall of the Ute Pass Thrust. Today the Fountain Formation, including the basal marine Glen Eyrie Shale Member, forms large flatirons, or inclined red rock beds. The Glen Eyrie Shale Member contains variegated shales, sandstones, and limestone of both marine and transitional environments. These facies change abruptly near the northwest-trending Ute Pass Fault as it abruptly grades into continental alluvial fans of the Fountain Formation.

The Ancestral Front Range Highland was then eroded and buried by more than 8,000 ft (2,440 m) of Paleozoic and Mesozoic sediments. As the mountains eroded down, the detritus was deposited in a series of high-energy flow regime braided streams to low-energy near-level plains, which are represented by the Fountain and Lyons Formations and later Mesozoic deposits. The Permian Lyons Sandstone (230 ft thick [70 m]) is partly eolian and partly fluvial sandstone. It consists of finer-grained nonmarine and marginal marine clastic rocks. Today the uplifted Lyons Sandstone and Fountain Formation comprise the striking bedrock fins in Garden of the Gods.



ERA	PERIOD	EPOCH	Colorado Springs Strata	
Cenozoic	Quaternary	Holocene	Piney Creek Alluvium	
		Pleistocene	Broadway Alluvium	
			Louviers Alluvium	
			Slocum Alluvium	
			Verdos Alluvium	
			Rocky Flats Alluvium	
			Nussbaum Alluvium	
	Tertiary	Pliocene	--------------	
		Miocene	Ogallala Fm	
		Oligocene	Castle Rock Conglomerate	
		Eocene	Wall Mt. Tuff	
		Paleocene	Dawson Formation	$Tkda_3$
				$Tkda_2$
				$Tkda_1$
Mesozoic	Cretaceous	Lower Dawson--Kda		
		Laramie Formation		
		Fox Hills Sandstone		
		Pierre Shale		
		Niobrara Formation	Smokey Hill Mbr	
			Ft. Hays Limestone	
		Graneros Shale, Greenhorn Limestone, Codell Sandstone, Carlile Shale		
		Dakota Sandstone, Purgatoire Formation		
	Jurassic	Morrison, Ralston Creek formations		
	Triassic	Lykins Formation		
Paleozoic	Permian	Lykins/Lyons Sandstone		
	Pennsylvanian	Lyons Sandstone/Fountain Formation/Glen Eyrie Shale		
	Mississippian	Leadville/Williams Canyon Limestones		
	Devonian/ Silurian	-------------		
	Ordovician	Harding Sandstone/Manitou Limestone		
	Cambrian	Peerless Dolomite/Sawatch Sandstone		
Precambrian	Proterozoic	PikesPeak Granite		
		Gneiss, Schist		

Figure 2. Stratigraphic Column and Correlation Chart. Note this figure is compiled from geologic maps completed by the Colorado Geological Survey in the Colorado Springs area, 1999-2003.

Mesozoic

The Mesozoic Era was generally less eperiogenic. Meandering streams deposited muddy river sediments flanked by swamps in which a variety of plants and reptiles lived. This environment is represented by the Lykins Formation, a non-marine aqueous shale unit. Overlying the Lykins is the colorful Jurassic Morrison and Ralston Creek Formations, which crop out near Garden of the Gods. These formations are reddish-brown, green, white to brown claystone with interbedded siltstones, sandstones, and gypsum. They represent fluvial and swampy continental environments.

About 85 mya the Western Interior Seaway expanded across Colorado, advancing from northeast to southwest. As a result, continental and near-shore Lower Cretaceous Dakota Sandstone and Purgatoire Formations were deposited. These rocks represent near-shore siltstones, claystones, and sandstones, with thin carbonaceous shales and minor coal beds that were deposited in fluvial, tidal-flat, and beach environments. The Dakota Sandstone forms a prominent 'hogback ridge' along the entire Front Range, but is mostly absent in Colorado Springs due to faulting. Some of the world's greatest dinosaur remains are found in the Morrison Formation and Dakota Sandstone between Denver and Canon City. Several dinosaurs were first discovered in Colorado, including the state fossil, *Stegosaurus*.

Overlying the Dakota Sandstone is the Carlile Shale, Greenhorn Limestone, and Graneros Shale, which were deposited in a warm, shallow sea. At the top of this unit in the Colorado Springs area is the Codell Sandstone Member. It is a light brown to white sandstone that forms a depositional shoreface platform on which the shallow limestone and chalk of the Niobrara Formation disconformably rests. The Niobrara Formation consists of two members: the Smokey Hill Shale and the underlying Fort Hayes Limestone and also forms a prominent hogback west of Colorado Springs. Recurrent fault-block movements of Precambrian age basement faults (Weimer 1980) controlled the thickness and areal extent of these formations.

Overlying these near shore deposits are widespread shallow marine shale deposits. In Colorado Springs this formation is called the Pierre Shale and forms most of the flat-lying bedrock throughout central and southern Colorado Springs. This 4,500-ft-thick (1,370 m) unit also forms the easily eroded slopes along the base of the Front Range. The Pierre Shale marks a long period of sea level rise and transgression of the sea across Colorado. In Colorado Springs the Pierre Shale has abundant ammonite fossils and numerous bentonite beds that are typically 1-3 in. thick (25 to 76 mm) (Carroll and Crawford, 2000).

After Pierre Shale time the Western Interior seaway slowly retreated to the northeast in response to large orogenic movements in Utah. As sea level lowered, a long intertidal beach front was formed. The beach deposits are known as the Fox Hills Sandstone along the Front Range. Coal, clay, sandstone and siltstone of the Laramie Formation were deposited in freshwater swamps behind the beach front. The Upper Cretaceous Laramie Formation is stratigraphically the lowest non-marine synorogenic sedimentary unit along the Front Range (Raynolds, 1997). Coal beds in the basal Laramie Formation have been mined for coal in the area. Overlying this strata the Upper Cretaceous and Paleocene Dawson Formation consists of discontinuous beds of light gray to yellow-brown sandstone and claystone of terrestrial origin with abundant fossil wood. The boundary between the Cretaceous and the Tertiary in Colorado Springs can be seen in the hillside at Popes Bluff. These sediments were deposited on a gently sloping surface of low relief in a climate warmer and wetter than today. The Dawson Formation is loosely consolidated and is finer-grained and thins eastward away from its source area.

Cenozoic

Major uplifting along the Front Range marks the early Tertiary. Large basement blocks of Proterozoic rock were uplifted and shortened by reverse faults during this time, called the Laramide Orogeny (Sonnenberg and Bolyard, 1997). Debate rages over the origin of the compressional forces but one important theory states that the subduction of the Farallon oceanic plate under western North America changed to a very shallow angle, there by remotely affecting Colorado. This drove subduction eastward and caused the uplifts and plutons in the Rocky Mountain region.

About 70-mya continental crust weakened by melting of an oceanic slab underneath, began to buckle and shorten. A series of great uplifts occurred as basement rock was carried to the surface. Some uplifts were simple domes, some were elongate folds, and most were bounded by a combination of folds and inclined block faults along which slabs of rocks were moved several miles. Long debated by geologists, details on the structural development of the mountain front have several possible solutions. One is that opposing thrust faults occur on both sides of the Front Range, which may account for shortening the distance between Grand Junction and Colorado Springs by about 35 miles (56 km).

During Laramide deformation the Front Range moved vertically with respect to the mountain flanks, over a period of 25 million years in the late Mesozoic and earliest Cenozoic time (Figure 3). Today Laramide faults are exposed as splays of the Rampart Range and Ute Pass Thrust Fault systems. South of Colorado Springs the Ute Pass Thrust loses displacement rapidly and the structure of the Front Range becomes an easterly dip slope south of Cheyenne Mountain.

Coincident with uplift of the Front Range was the structural development of the Denver Basin. At its greatest depth near Castle Rock the basin is composed of 9,000 ft (2,740 m) of Cretaceous and lower Tertiary sediments (Raynolds, 1997). Material eroded from the rising highlands accumulated in flanking basins and spread outward as extensive fans of sediment on the plains. These strata reflect at least two discrete episodes of active thrust movement on the Front Range bounding faults with uplift in Colorado Springs.

Tertiary intrusive rocks were emplaced as the Laramide uplifts rose in response to crustal shortening. This zone runs across the state from southwest to northeast and is known as the Colorado mineral belt. It contains most of the state’s deposits of gold, silver, lead, and zinc and is located 60 miles (96 km) west of Colorado Springs.

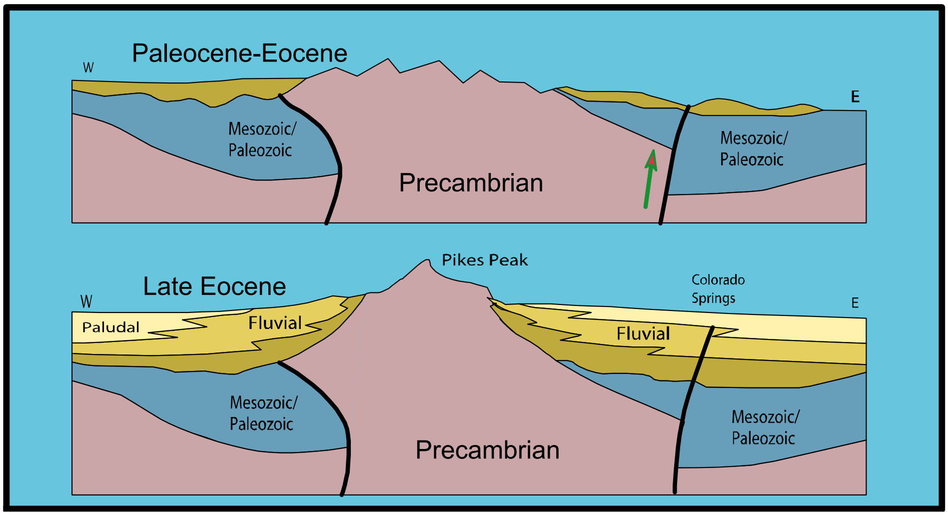


Figure 3. Diagrammatic representation of Laramide Structural Style in Colorado.

Laramide uplifts ended about 45 mya, but erosion continued. By 35 mya mountains were eroded to a series of hills and low isolated mountains that rose above a rolling plain that were only a few thousand ft above sea level. Remnants of this erosion surface (Rampart Range area) are the oldest record of Laramide mountain building preserved in the modern landscape (Steven and others, 1997). A Late Eocene surface of generally low relief developed on Precambrian rocks (Epis and Chapin, 1975). During the Oligocene, major volcanic eruptions in the Collegiate Range deposited extensive welded tuffs to the east across the Denver Basin. Many of the volcanic deposits choked existing fluvial streams creating paludal deposits preserving fossil trees, plants, and insects near Florissant. Later dissected by modern stream erosion, remnants of the volcanic events can still be found north of Colorado Springs. Known as the Wall Mountain Tuff and the Castle Rock Rhyolite, these rocks cap solitary buttes north of Monument (Johnson and Raynolds, 2002) and to the west.

In early Miocene time, uplift and block faulting caused further disruption of the drainage system created after volcanism. Basin and range block faulting in the Miocene and Pliocene resulted in deep incision of the old surface, and related volcanism ended about 20 mya. Up to 1,000-ft (305 m) of Tertiary rock material may have covered Colorado Springs at that time. Oligocene and Miocene deposits were later removed by erosion in Colorado Springs, and deposited in eastern Colorado as the Ogallala Formation. The Front Range landscape evolved through a succession of structural developments and are preserved in the facies distribution patterns reflective of the interplay between sediment supply and accommodation spaces (Raynolds, 1997).

In the last 26 million years extensional tectonic forces have pulled at the Western US, and fault blocks again moved. The Southern Rocky Mountains and western Great Plains Provinces were uplifted to over a mile high. Extensive erosion sculpted the broad outlines of present-day topography. Uplifted canyon cutting and a wetter climate combined to erode the mountain front and deposit broad alluvial material on the eastern plains (Steven and others, 1997). More resistant rocks such as the Pikes Peak area remained higher than the more erosive sediments of the Denver Basin.

Quaternary

The Quaternary Period (last 1.8 million years) is marked in Colorado Springs as one of glacial carving and stream erosion from the mountain front. The latest climatic shifts in the Quaternary are marked by deposition of glacial moraines and outwash terraces, pediment gravels, and Holocene fluvial deposits. In Colorado Springs four older pediment gravels capping eastward inclined surfaces represent deposits from these events. Nussbaum Alluvium is the oldest Quaternary pediment in Colorado Springs and is located on isolated mesas near Monument, and is considered earliest Pleistocene in age. Rocky Flats Alluvium pediment gravel originated as alluvial fans deposited by streams draining the Front Range. Thick pedogenic calcium carbonate occurs in the upper parts of these units. Considered Nebraskan or Aftonian in age, they consist mostly of hard quartzite boulder gravels. Lower, younger pediment gravels are the Verdos and Slocum Alluviums. These pediments are similar to Rocky Flats, but contain fewer boulders and no carbonate soil. The Lava Creek B ash-fall is interbedded in the Verdos Alluvium, from which a minimum age of 620,000 years was derived (Scott and Wobus, 1973). The Rocky Flats, Verdos, and Slocum Alluviums (or Qg3, Qg2, and Qg1, respectively, as mapped by the Colorado Geological Survey) form prominent gently dipping pediments of reddish-brown granitic cobble gravels that steepen toward the mountain front (Carroll and Crawford, 2000). Formed in response to climatic change these alluviums reflect major glacial changes in the Rocky Mountains.

Lower on the valley sides are three less prominent alluvial stream terraces of the modern drainage systems for Cheyenne Creek, Monument Creek, and Fountain Creek. These low surfaces are less than 50 ft (15 m) above the modern stream drainage and are composed of clasts typical of the modern stream drainages. These are the Louviers, Broadway, and Piney Creek Alluviums, or locally mapped as Qt3, Qt2, and Qt1 (Carroll and Crawford, 2000). Modern stream channels along Monument and Fountain Creeks have deposited Holocene terraces along the creek edges. Wind blown deposits of sand are abundant east of Monument Creek. Total alluvial and eolian deposits sometimes cover bedrock up to 150-ft thick (46 m). Much of the modern hillsides in Colorado Springs are covered with residuum and colluvial material. Many landslides cover areas in Colorado Springs susceptible to large-scale earth movements.

The youngest deposits are those human-made deposits of artificial fill and mining mill tailings (Golden Cycle Mill). These deposits, along with concrete and asphalt highways, dams, grassy slopes, buildings, and other structures, cover the surfaces of most urban parts of Colorado Springs.

GEOTECHNICAL CHARACTERISTICS

The geotechnical characteristics of the surficial and bedrock units in the Colorado Springs area can change dramatically over very short distances. The following general descriptions provide typical conditions that can be expected for specific rock and surficial units (Plate 1) from youngest to oldest. Under each unit are described the general geotechnical properties and more common geologic hazards. Many of the geologic constraints can be controlled through sound engineering design. On the other hand, areas affected by serious geologic constraints, such as active landslides and rockfalls are best avoided.

Foundation Related Geologic Units

The pediment gravels and alluvial terraces consist of boulders, cobbles, gravel, sand, silt, and clay (Costa and Bilodeau, 1982; Bilodeau et al, 1983; and Cochran, 1977). These deposits generally provide sufficient foundation support for most structures. Locally, isolated lenses of expansive clay, from 1 to 5 ft thick (0.3 to 1.5m) are present near the ground surface. A caliche layer at depths of 2 to 6 ft (0.6 to 1.9m) is commonly present in the older deposits that have well-developed soils. Heaving-soil problems maybe encountered in areas of swelling clays. Mass movements can occur along steeper hillside slopes or along undercut areas. These deposits are generally excavated using heavy-duty equipment.

Eolian sand deposits generally occur east of Monument and Fountain Creeks and usually provide good foundation support (Cochran, 1977). However, some of these deposits have scattered areas with low to moderate collapse potential (1 to 4 percent). Collapse prone eolian soils have been encountered in the Fountain and Colorado Center areas, and in the vicinity of Academy Boulevard and Austin Bluffs Parkway. The deposits are generally excavated using standard techniques.

The "Qrof" deposit mapped by Scott and Wobus (1973) along the Cheyenne Mountain front frequently contains large boulders (up to 20 to 30 ft, 6 to 9 m) with gravel, sand, silt, and clay matrix. Closer to the mountain front the unconsolidated deposits can be locally clast supported, but are generally matrix supported (by gravel and finer material). This material generally requires heavy-duty equipment for excavation and fragmenting of larger boulders for removal.

The Upper Dawson Formation covers large portions of northern and eastern Colorado Springs. It is composed of conglomerate, sandstone, siltstone and claystone. Generally the formation provides adequate bearing strength for most structures. The claystone and clayey sandstone layers and the soils that develop from them generally have low to very high swell potential. The claystone and many of the sandstones are usually easily excavated, but can require significant effort to breakdown and mix for use as fill. The conglomerate and cemented sandstone layers often require ripping for excavation. In northeastern Colorado Springs beds of weathered, iron-oxide cemented sandstone have shown elevated low-energy gamma radiation source materials (more than twice regional background).

The Upper Cretaceous Lower Dawson (formerly Arapahoe) Formation is generally exposed along the bluffs on the north and northeast sides of Colorado Springs. It is composed of conglomerate, sandstone, shale and claystone. Generally, this formation provides adequate bearing strength for most structures. The claystone is generally expansive and will swell when moisture is introduced. The shale and claystone are generally easy to excavate, while the conglomerate and sandstone may require ripping.

The Laramie Formation consists of interbedded shale, siltstone, claystone and sandstone that contain thick lenses of coal and clay. The coal from this formation was extensively mined beneath the Rockrimmon, Colorado Springs Country Club, and Cragmor areas. Subsidence over abandoned mines is a continuing problem in some areas. Slope stability can be an issue in areas with claystone beds near the surface. Laramie sandstone beds frequently form cliffs or bluffs in northern Colorado Springs, creating rockfall hazards locally in the Popes Bluff, Rockrimmon, Pine Cliff, and Woodmen Valley areas. Excavation of the sandstone is difficult. The clay, claystone and shale beds are generally expansive and in some areas are highly to very-highly expansive. Foundations on the Laramie Formation generally perform well, provided proper engineering solutions to the expansive materials are utilized, and that areas underlain by shallow coal mines are avoided.

The Fox Hills Sandstone is composed predominantly of sandstone and siltstone, and varies from uncemented to very hard. The formation generally has high bearing strength. Excavation of the siltstone beds is relatively easy, but where cemented the sandstone is very difficult to excavate.

The Pierre Shale consists of shale with thin bentonitic, siltstone and sandstone layers. The upper portion of the Pierre Shale is more silty and sandy. Where exposed, the Pierre Shale generally weathers to clay. The swell potential is generally highest in the weathered claystone and clay. However, the unweathered shale and all of the weathering products can have swell potentials ranging from low and very high. Generally dip slopes between about 15 and 45 degrees are unstable. This is primarily due to the frequent bentonitic layers that have low shear strength properties. Landslides overlying Pierre Shale can be quite large: some covering 100's of acres (100's of hectares) in size have been identified.

Due to a combination of open spaces, parks, and faulting and folding along the mountain front other Cretaceous, Jurassic, Triassic, and Permian age formations are generally not widely subject to development. Where the sandstones are exposed they generally form ridges or spires with the shale and siltstone forming the adjacent slopes. These formations generally have foundation characteristics similar to the Laramie Formation.

The Fountain Formation is generally exposed within, west, and north of Garden of the Gods Park. The formation locally consists of interbedded lenticular conglomerate, sandstone, siltstone and shale layers. In the Glen Eyrie Shale Member, the shale beds are more frequent and generally greater than 5 ft thick (1.5 m). Some of the beds within the Glen Eyrie Shale Member are highly plastic and have low shear strengths and moderate to high swell potential. The exposures of the Fountain Formation within, west, and north of Garden of the Gods Park generally are dip slopes to the east, with numerous landslides in areas with exposed shale. Foundation conditions on Fountain Formation vary depending on the nature of the underlying bedrock and adjacent slopes. Rockfall hazards from the cliff forming sandstones exist locally in Manitou Springs and Garden of the Gods Park.

Limestones and dolomites of the Mississippian and Ordovician age exposed in Manitou Springs and western Colorado Springs north of Manitou Springs are generally good foundation materials, but are difficult to excavate. The limestones and dolomites are subject to karst dissolution, as observed at the Cave of the Winds. The silica-cemented sandstones of the Sawatch Sandstone are good foundation materials, but can be difficult to excavate.

Crystalline Precambrian rocks exposed west of the Rampart Range and Ute Pass Faults present few foundation problems beyond excavation and slope stability. However, some of the granitic rocks have elevated levels of low-energy gamma radiation, and there are scattered adits from mining in the late 1800's.

Exploration and Testing Methods

Exploration methods used to define the surface and subsurface conditions at potential building sites include review of the geologic literature available, site-specific geologic hazards evaluations, followed by drilling, test pits, trenches, or a combination of the three. During the subsurface investigation samples are usually taken at regular intervals or at apparent changes of materials, and tested to determine their engineering properties. Field and laboratory tests are usually performed in general accordance with American Society for Testing and Materials (ASTM) or American Association of State Highway and Transportation Officials (AASHTO) testing methods by in house laboratories. Trained technicians under the direction of a geotechnical engineer generally perform testing.

The most common subsurface drilling technique used locally is solid-stem continuous flight power augers. Occasionally hollow-stem power augers are utilized. The primary sampling method is with driven modified California barrels, which are thick-walled split barrel samplers with approximately 2-in-diameter (51 mm) by 4-in-long (102 mm) brass liners. Standard split-spoon samplers are sometimes used for noncohesive materials. Push sampling with thin-wall samplers is generally not recommended, because the local fine-grained soils are too stiff. Occasionally the bedrock is cored using either air- or mud-rotary methods.

Common tests performed include Atterberg limits, grain size distribution, moisture content, dry density, one-dimensional swell-consolidation, standard and modified Proctor moisture density relationships, unconfined compression, soluble sulfates and pH, and occasionally direct shear or triaxial shear. The primary design test methods used by most local geotechnical engineers for swelling soils are the one-dimensional swell-consolidation or Federal Highway Administration.

Swell-consolidation tests are usually performed in general accordance with ASTM D4546, Method B, and typically plotted as shown in Figure 4. Locally-derived materials are generally considered to have low, moderate, high or very high swell potential if the swell is between 0-2, 2-4, 4-6, and over 6 percent respectively under a confining pressure of 1000 pounds per square ft (0.488 kg/cm^2) when water is added to the sample (CAGE, 1996). Swelling potential is discussed above under characteristics of the individual foundation bearing geologic units.

Foundation Types

Typical foundations in the area are spread footings, footing pads with grade beams, drilled piers with grade beams, and post-tensioned slabs (see Costa and Bilodeau, 1982; Bilodeau et al, 1983; and Noe et al, 1997 for diagrams). In the past, bearing walls on grade were used. Spread footings are most commonly used for residential and low-rise commercial buildings. Drilled piers are generally only used for larger commercial buildings and some residential foundations on sites with swelling or collapsing soils. The majority of residential structures in the Colorado Springs area have basements under at least a portion of the house.

The most common foundation type on swelling soils involves excavating 4 to 10 ft (1.2 to 3 m) of soil below the bottom of the foundation, replacing the removed soil with compacted granular fill, and placing a spread footing foundation on the granular fill. This excavation and replacement is normally accompanied by the use of slab-on-grade floors. The depth of typical foundations in Colorado Springs appear to provide sufficient soil cover so that most residences do not suffer from the effects of seasonal moisture variation movements. On sites with swelling soils, where drilled piers are used, the lowest level floors are typically structurally supported to

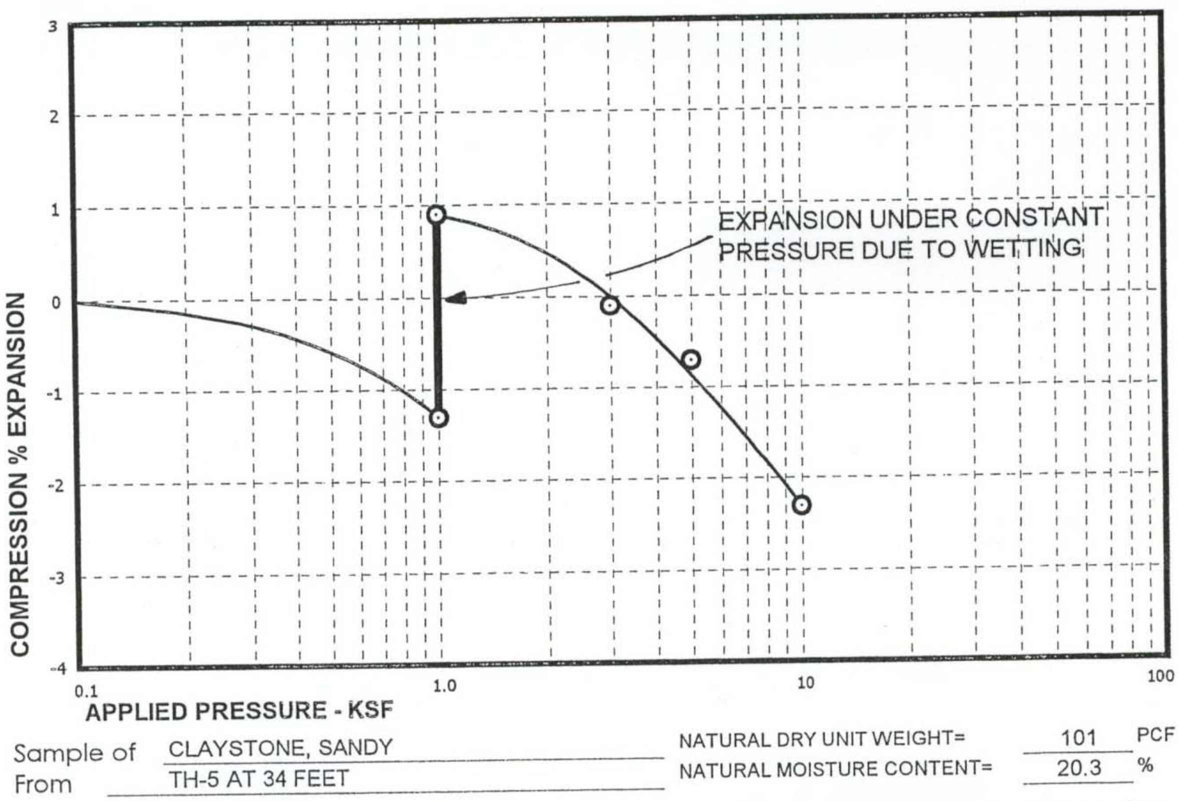


Figure 4. Typical swell-consolidation test plot.

provide a void space for the swelling soils to expand into without distorting the floor. Post-tensioned slabs are being used frequently for apartment and multifamily buildings.

In areas of steeply dipping (dip greater than 30 degrees) (Himmelreich and Noe, 2000; Noe, 1997; Noe et al, 1997) expansive bedrock, foundation construction usually consists of excavating and replacing the materials to a depth of at least ten-foot below the bottom of the foundation as shown on Figure 5. This ten-foot buffer of compacted material has been effective in the Denver area and has been adopted by the Colorado Spring geotechnical community.

Some collapse prone soils have been recognized locally. Thin near surface layers of these soils generally collapse during site grading. Where they have been recognized they have typically been excavated to a depth of 4 to 6 ft (1.2 to 1.8 m) and replaced with compacted granular fill, and spread-footing foundations used to support the structure.

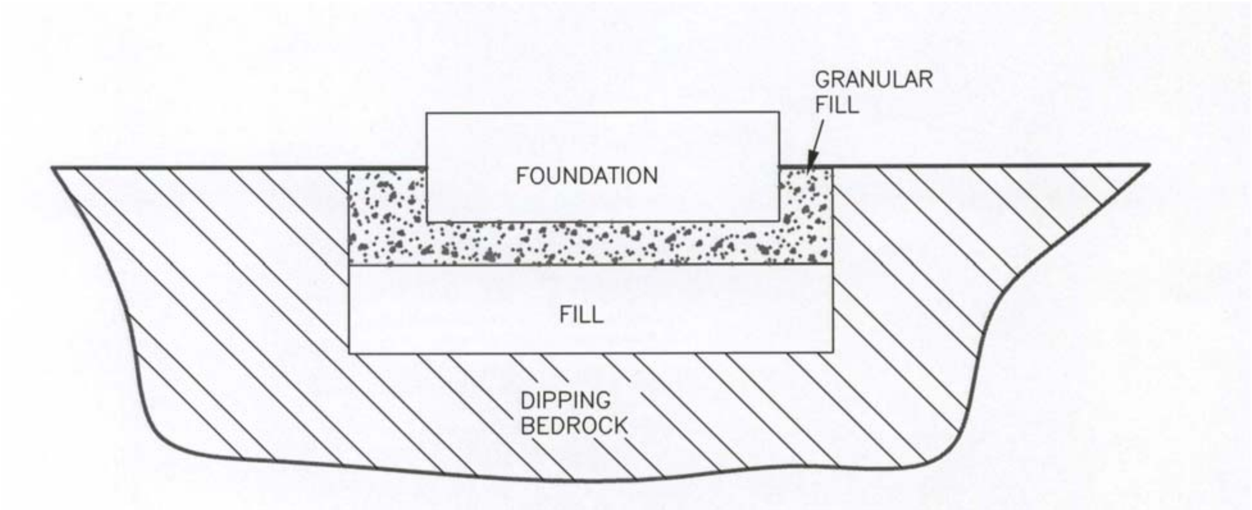


Figure 5. Typical subexcavation for footings in dipping bedrock.

MATERIALS

Aggregates

The Colorado Springs area has abundant high quality sand and gravel aggregate. The majority of the sand comes from the alluvial-terrace sand deposits along Fountain, Monument, Sand and Jimmy Camp Creeks, and the gravel comes from pediments and fan deposits. Additionally the eolian sand deposits east of these drainages have provided abundant fine material for concrete and asphalt mixes, and for use as structural fill. There are 105 permitted sand and gravel operations and six permitted general borrow pits in El Paso County (Colorado Department of Natural Resources, Division of Minerals and Geology, 2002A). Empire Laboratories performed an aggregate resources survey for El Paso County in 1991 (Empire Laboratories, 1991), and identified the best-quality sources from the aeolian sands east of Colorado Springs for fine aggregate, the pediment and Fountain Creek alluviums south of Colorado Springs and the limestone west of Colorado Springs for coarse aggregates.

Production of some crushed rock aggregates from bedrock sources has occurred in western and northwestern Colorado Springs. The majority of the aggregate from these pits has been used for concrete and asphalt.

Colorado Silica Sand and other companies have been producing high quality (consistently-sized) sand for many commercial uses for years from the eolian sand deposits on the northeast side of Colorado Springs. The sand has been used in bulk for sand blasting, frac sand for oil and gas production, sand cone density sand and other applications where high quality uniform size rounded silica sand is required. There are about 15 permitted silica sand mines in El Paso County (Colorado Department of Natural Resources, Division of Minerals and Geology, 2002A).

Clay

There are four permitted clay pits and one shale pit in El Paso County (Colorado Department of Natural Resources, Division of Minerals and Geology, 2002A). The clay is mined primarily for the manufacturing of bricks for use locally in construction.

Limestone

There are four permitted limestone quarries locally in the western parts of Colorado Springs (Colorado Department of Natural Resources, Division of Minerals and Geology, 2002A). The permits for the mines have been active with the current operators since 1980. The limestone is used primarily for concrete aggregate in the local area.

Coal

A portion of northern Colorado Springs is underlain by subbituminous coal found in the Laramie Formation. There were 74 operating coal mines in El Paso County between 1883 and 1965. Of these, 65 were located in northern and eastern Colorado Springs, leaving portions of Colorado Springs undermined (Carroll and Bauer, 2002). The locations of the undermined areas of Colorado Springs are shown on Figure 6. Some of the areas where the coal mining was relatively shallow, less than 100 ft (30m), have experienced significant subsidence (Colorado Department of Natural Resources, Division of Minerals and Geology, 2002A). Generally, areas where the mining occurred more than 100 ft (30 m) below the ground surface have not experienced as much subsidence.

GEOLOGIC CONSTRAINTS

Geologic Hazards Evaluations

Colorado Springs has a number of geologic constraints that may pose hazards to engineered works. Many of these have long been recognized. Investigators from the U.S. Geological Survey and CGS (Scott and Wobus, 1973; Hart, 1974; Trimble and Machette, 1979; Carroll and Crawford, 2000; Thorson et al, 2001) described some of the landslide deposits and other hazards related to the geology in their maps. After passage of Colorado House Bill 1041 advising counties and municipalities to define and locate geologic hazards, Charles Robinson and Associates (Cochran, 1977) was hired to complete a comprehensive mapping study of El Paso County that included engineering geologic mapping. El Paso County's land development code was modified to include geologic hazards within a few years of the legislation. The City of Colorado Springs enacted a Geologic Hazards Ordinance in 1996 in response to a landslide in an affluent area that distressed or severely damaged 5 homes during a wet spring and summer in

1995. The City of Colorado Springs drew on local and State experts when they enacted their Geologic Hazards Ordinance. Manitou Springs also adopted a geologic hazards section during an update of their land development code.

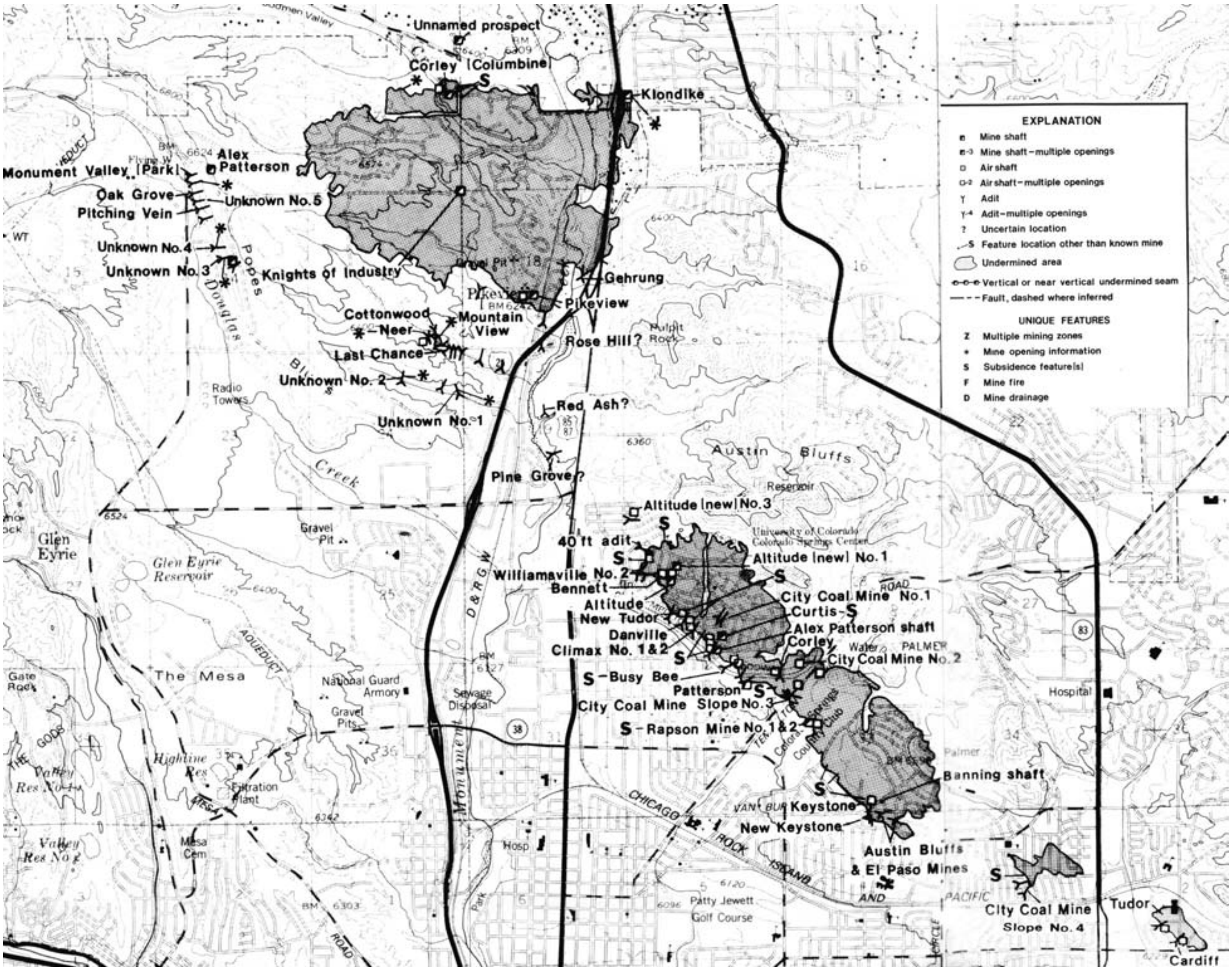


Figure 6. Areas of Colorado Springs underlain by Coal Mines (from Carroll and Bauer, 2002).

The City of Colorado Springs Geologic Hazards Ordinance requires annexations, development plans, rezoning, and new construction, including additions, to have a geologic hazards evaluation performed and submitted to the City as part of the development process (City of Colorado Springs, 1999). The geologic hazards ordinance outlines items to be reviewed and included in the submitted reports. These items include: descriptions of site evaluation techniques used, bedrock units, surficial units, geomorphic features, structural geology features, surficial drainage, groundwater, geologic interpretation, impacts on development and intended use of the property, conclusions and recommendations. The list of geologic hazards in the City's ordinance to be addressed in the geologic hazards reports is comprehensive and includes: steep slopes, landslides, rockfall, swelling soils, steeply dipping swelling bedrock, collapsible soils, mine subsidence, shallow ground water, radon gas, debris and mud flows, and earthquakes. The ordinance provides a valuable resource and guide for both the types of geologic hazards in the Colorado Springs area and methods to be employed to further identify and characterize these hazards.

The geologic hazards discussed here are those that have been identified and analyzed by the authors while meeting the requirements of the ordinance during site characterization for development at numerous sites. They are discussed in the order that they appear in the city ordinance. Not all geologic conditions that may be hazardous are discussed and there may be certain conditions that will prove to be hazardous to future engineered works that have not been identified at this time.

Landslides and Potentially Unstable Slopes

Landslides include slides, lateral spreads, and complex landslides as described by Varnes (1978). Potentially unstable slopes are defined as those slopes that in their current configuration are stable, but any modification to the slope through site grading, increase in water content, or erosion may cause the slope to become unstable and may initiate a slope failure. Identification of these slopes and their engineering characterization can be difficult. There are many past examples where development without adequate mitigation has impacted potentially unstable slopes with devastating results, as shown in Figure 7.

Structurally, the sedimentary rock sequence in Colorado Springs has a northeasterly dip. The result of this structural orientation is that many areas of potentially unstable ground exist along north and east facing slopes due to potential failure zones along bedding planes.

If the dip of the bedrock or the type of material is not considered when developing a site slope stability problems can develop. Slope failure can be initiated through site grading by removing support at the base of the slope, or increasing the loading at the top of the slope. Poor site drainage, irrigation, and episodes of severe rainfall can lead to an increase in water content in the slope. Water is a significant factor in decreasing the strength of slopes and creating mass movement within them.

Slope stability analysis is generally recommended where dipping geologic structure or weak materials and sloping ground coexist. This is especially true where dip slopes of expansive bedrock are exposed. If avoidance of these areas is not an option, development generally requires extensive engineering mitigation. Mitigation alternatives include buttresses, rockbolts or anchors, engineer-designed retaining walls with proper drainage systems, or removal of unstable rock or soil masses. Construction and engineering personnel should be aware of dipping bedrock hazards and the areas where it occurs so that proper identification mitigation measures can be taken. Because of site disturbance and grading restrictions in city ordinances not all of these options are available in many part of the city (City of Colorado Spring Planning Department, 2002).

Landslides are quite common in Colorado Springs primarily on north-facing slopes of the Pierre Shale. Up to twelve individual slides impacting homes and development infrastructure occurred in the Colorado Springs areas during and after heavy rainfall in 1999. Many of these were small, thin-skinned events, but others, such as the one at Holland Park subdivision (Thorson et al., 2001), were deep-seated rotational landslides that left up to eight acres of residential land unusable. Evidence of past instability associated with easterly dipping bedrock is found on the west side of Colorado Springs where many ancient landslides exist. Although many of these

Figure 7 A (above). Former parking lot for auto dealership at the head of landslide. Figure 7 B (below). Toe of the same landslide encroaches onto auto dealership parking lot (Photos by K. Andrew-Hoeser).

slides may not be currently active these materials are weak and susceptible to repeated sliding. With the increasing development in Colorado Springs, many portions of these older landslide deposits have been reactivated. The most heavily impacted area has been in the Broadmoor Bluffs (southwestern Colorado Springs). Many of the older and more recent landslides have been mapped by Carroll and Crawford (2000), Thorson et al (2001), and Rowley et al (in press). The Colorado Geological Survey conducted a study for the City of Colorado Springs addressing currently active landslides and the homes damaged or threatened by these slides in 1999 to meet Federal Emergency Management Agency requirements for subsidized purchase of the properties (Colorado Springs Utilities, 2000). White and Wait (in publication) are producing a map of City of Colorado Springs that shows locations of identified landslides and general areas they consider susceptible to landslides.

Rockfall Hazards

Several bluffs that are attractive natural geologic settings for homesites exist in the Colorado Springs area. However, rockfall is a hazard to developments located below the bluffs and may cause loss of support for structures above them. Rockfall hazards exist wherever cliffs and large rock fragments are located on slopes and are subject to becoming detached and toppling, falling, rolling, or sliding down the slope (Rogers *et al* 1974; Shelton and Prouty, 1979). Figure 8 shows the range of rockfall types typically encountered in the Colorado Springs area.

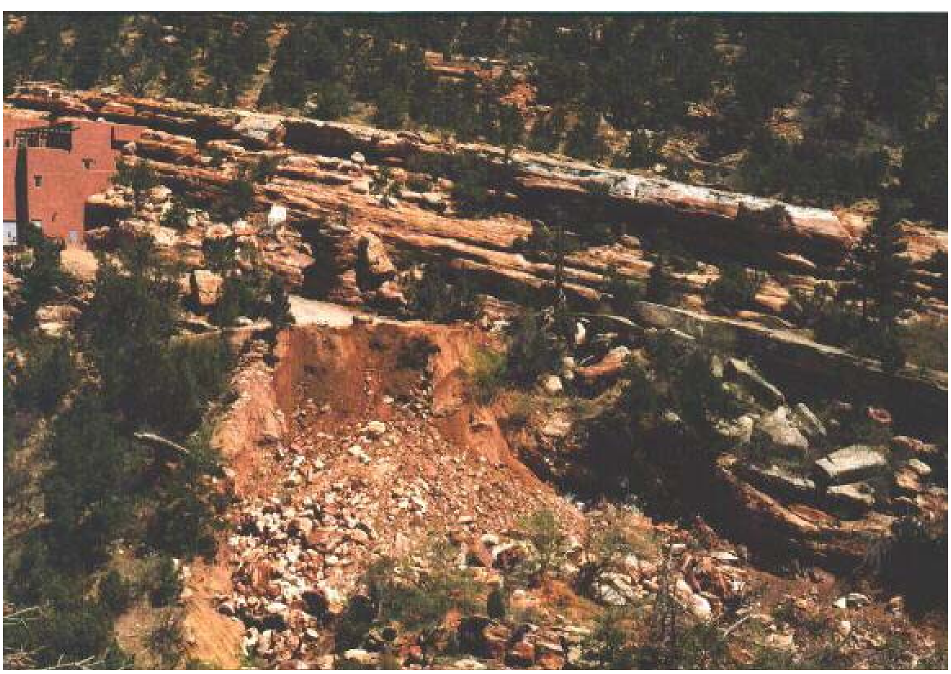


Figure 8. Rock slide, with topples above and to sides, at Manitou Cliff Dwellings (Photo by W. Hoffman).

Several areas, particularly in the northern and western portions of Colorado Springs and Manitou Springs, have exposed sandstone and limestone cliffs and hogbacks where boulders and cobbles are produced by erosion and create various degrees of rockfall hazards. Generally these levels can be mapped as three zones. The description of these zones and typical mitigation recommendations are as follows:

Source Area – This is the cliff or rockfall source area that produces the rock fragments. This area carries the highest risk for rockfall damage and is usually avoided by development. It can be stabilized to eliminate or reduce the risk to rockfall in the zones below. Stabilization can include scaling, or removal of loose rocks prior to construction. It can include techniques designed to hold the loose rocks in place and prevent them from becoming dislodged, such as of rock bolts to bolt the rocks to the cliff face, anchored wire mesh, cable lashings, non-shrink grout to support the undersides of loose rocks, and shotcrete for containing small loose rocks subject to erosion (White, 1996). Some of these techniques, although effective can be aesthetically unattractive.

Runout Zone, High Velocity – This area is the runout zone immediately beneath the source area. Structures in this zone may be impacted by boulders having moderate to high velocity. If construction in this area cannot be avoided, typical mitigation involves stopping the boulders by use of fences, walls or earthen berms designed for predicted rock sizes and behavior (White, 1996).

Runout Zone, Low Velocity – This area is the lowest runout zone where boulders are moving at a slower velocity and coming to rest. Mitigation measures in this area involve those used in the high velocity zone, but designed to stop slower rocks.

These zones can be evaluated visually (Rogers et al., 1974) or by the use of the Colorado Rockfall Simulation Program (CRSP) (Andrew, 1996; Jones et al., 2000) or other similar programs. CRSP predicts rockfall behavior given a specific set of boulder characteristics and slope conditions at any point on the slope. CRSP can also predict rock velocity and bounce height for use in design of catchment structures such as fences, walls or berms at any point along the slope. Judgment must be exercised using the program as its results are not absolute.

Although areas of rockfall hazard can be mitigated, the stabilization measures require periodic observation and maintenance. Erosion is an on-going process and previously stable rocks can become dislodged and unstable. Catchment structures require periodic cleaning to remove built up debris.

Swelling Soil and Bedrock, and Collapsing Soils

Swelling and collapsing soils have caused damage in the Colorado Springs area costing millions of dollars. In comparison, however, the expansive soils problems have not been as extensive or widespread as in the Denver area. Colorado Springs has been relatively fortunate that its rapid growth has been in areas where expansive bedrock generally did not occur until reasonable engineering solutions for the problems were developed. These solutions are described above in the foundations section.

In expansive soils the clay minerals absorb water, causing them to swell or expand when wet and to shrink when dry (Noe et al, 1997). This swelling and shrinking puts pressure on foundations, floors, sidewalks and drainage systems that, over time, can cause cracking and considerable damage. The primary formations that have expansive soils and bedrock are Pierre Shale, Laramie, Dawson, and the Glen Eyrie member of the Fountain.

Collapsible soils generally are fine-grained deposits with a meta-stable structure that have not been fully saturated with water since deposition. The structure of the soil is typically silt grains, bound to one another by clay or other minerals (especially gypsum) in an open, or void structure that has a high dry strength. Collapsible soils are often associated with wind-blown loess, which occur in Fountain, Colorado Center and in the vicinity of Academy Boulevard and Austin Bluffs Parkway. The collapse in these areas has generally been triggered by over irrigation and saturation of the soils. Collapsible soils are also found in the colluvium at the base of the Niobrara Formation. Natural successive subsidence scarps have been observed in these deposits of low-density silty clays (Figure 9).

Figure 9. Concentric subsidence features in low-density silty clays at the Base of the Niobrara Fomation (Photo by J. Lovekin).

Steeply Dipping Expansive Bedrock

Some of the most scenic areas of the Front Range and Colorado Springs regions are at the base of the mountain front where Paleozoic and Mesozoic sedimentary beds have been tilted and upturned by the uplifting forces that formed the mountains. A few examples include the Garden of the Gods Park in Colorado Springs; Perry Park, Roxborough State Park, and Red Rocks Park in the Denver area; and the Flatirons of Boulder. Within Garden of the Gods, the bedrock dip is vertical to near vertical. The more resistant sandstone and limestone beds form spectacular outcrops, while the less resistant shale and siltstones weather into valleys.

Garden of the Gods Park also provides a good illustration of the geologic conditions that create hazards associated with steeply dipping expansive bedrock. The more resistant rock forms the spires and is typically material that, from an engineering standpoint, has low or no expansion potential. The less resistant geologic material is typically expansive shale and siltstone beds that weather to the flat lower ground in the park.

Where expansive layers are near horizontal; a structure built over an area will encounter a single geologic unit with relatively constant expansion potential. Where expansive materials are structurally tilted and steeply dipping, as is the case in the Garden of the Gods Park, a number of individual beds with diverse expansion potential may be exposed over a relatively small area. Due to this, the expansion potential across a building pad, may vary considerably (Noe et al., 1997), as several geologic materials with widely different expansion potential underlie the structure.

Local Colorado Springs engineering firms have noticed that traditional measurements of expansion potential underestimate the actual forces and stresses imposed on a structure when the beds are tilted. This is caused by several factors: The upturned orientation of the bedrock and fractures from the folding facilitate the movement of water into the bedding planes, increasing the potential for saturation, and expansion. The different geologic units have widely varying expansion potentials, because of differing amounts and types of clay minerals in the different layers. Therefore erratic swell conditions may exist over relatively short horizontal distances.

The highest hazard is associated with beds dipping 30 degrees or greater. This zone has been approximately mapped by the CGS (Himmelreich and Noe, 2000), and much of it is already developed in Colorado Springs.

Typical mitigation for foundations built on expansive, steeply dipping bedrock in the Colorado Springs area is excavation and backfill as shown in Figure 5. Many times the clay soils can be used in the lowest portion of the excavation if mixed, broken down, and moisture conditioned to above optimum, and compacted. The upper few feet usually consists of granular fill. Mitigation is typically recommended in areas where steeply dipping, expansive bedrock is encountered within 10 ft (3 m) of the foundation level. Drilled piers are an option used in some areas of expansive soils, but are not recommended where the beds are dipping steeply.

Mine Subsidence

Mine subsidence has occurred in the northern parts of Colorado Springs, generally in areas that e underlain at depths of less than 100 ft (30 m) by coal mines. The majority of the subsidence has occurred over mines in the north central part of Colorado Springs in an arc from about Cragmor Road on the northwest to about Murray and Palmer Park Boulevards on the southeast. Mines in this area were generally at depths less than 250 ft (76 m) below the ground surface with some areas less than 50 ft (15m) (Dames and Moore, 1985). Some subsidence has also occurred in the

Rockrimmon area, but generally less damage has occurred in this area because the mining was generally at depths of over 150 ft (46 m).

The subsidence is caused by collapse of voids left behind after removal of the coal. The mines were generally excavated to the maximum width that the pillars and walls could support the overlying rock. Over time, the overlying rock has fractured in response to the increased stress, and the walls and pillars have weakened. This weakening and over stressing of the rock has lead to collapse of the mine workings. The collapse will stope upward until it reaches the ground surface or a rock layer has sufficient strength to bridge the underlying void.

The state of Colorado does offer mine a subsidence protection program for residences built prior to February 22,1989 within subsidence prone locales (Colorado Department of Natural Resources, Division of Minerals and Geology, 2002B). The program is not a standard insurance, but it does require an initial registration fee and annual participation fee. The program will pay up to $50,000 per occurrence. The program requires an initial inspection by the program's qualified engineering firm, and inspections for each claim with reports and estimates by the engineering firm.

Mine and Mill Tailings

There are several small and large tailing piles within Colorado Springs, including a massive one that is a dominant topographic feature. Most of the tailings in Colorado Springs are related to the mills that operated south of Old Colorado City from the 1890's to the late 1940's (Idleman and Aldridge, 2000). Early in their operation the mills received partially processed gold and silver ore transported by rail from mines in the Cripple Creek and Victor area. The mills were located in the area because the ore with a high sulfate content required coking as part of the smelting process, and coal was available locally from mines on the north side of Colorado Springs. The tailings from the mills were generally deposited in shallow ponds near by and the water was either recovered for reuse or allowed to evaporate. As the ponds filled, the dikes were typically raised with the coarse tailings utilizing the upstream raise method of construction. The largest of the tailings ponds (Gold Hill Mesa) eventually reached a height of about 200 ft (61 m) from the toe to the crest of the embankment. Due to the slope of the dam and the slope of the underlying original ground surface the maximum thickness of the tailings is estimated to be about 130 ft (40 m). The last operating mill and associated tailing pond was closed in the late 1940's (CTL Thompson, Inc., 2002).

To date erosion has been the biggest problem that the tailings have presented. A developer is currently in the early stages of reclamation of the largest tailings pile located south of Highway 24. The proposed development includes single and multifamily residential, parks, and commercial areas. The environmental aspects of the project are addressed though a voluntary cleanup plan, which calls for a soil cap over the exposed tailings (Vickers, 2003). The cap serves to reduce the exposure of the residents to the low level contaminants in the tailings and reduce the radon levels in the structures.

Radioactivity and Low-Energy Gamma Radiation Surveys

Rocks containing uranium have been recognized in both the Upper Dawson Formation in the Black Forest region and in the metamorphic and igneous Precambrian rocks west of Colorado Springs (Nelson-Moore et al, 1978). The two principal hazards associated with uranium-bearing rock are the gamma radiation emitted by the decay of the uranium and generation of radon gas a radioactive daughter by-product. The deposits in the Upper Dawson Formation were first recognized in a regional study for uranium in the mid-1970's, reported by the CGS (Thorson et al, 2001; Nelson-Moore et al, 1978; CGS, 1991). Concentrations of 0.52 percent U_3O_8 were measured from a uraniferous limonite in the $Tkda_3$ lithofacies of the Dawson Formation. This surface material typically consists of highly cemented, dark brown conglomeratic sandstone. The sandstone fragments are apparently remnants of a mineralized sandstone channel that has since been eroded, so that only scattered surficial "float" deposits remain. Some limited zones of elevated radiation have been measured at depth in the bedrock. Elevated low level gamma radiation readings have also been observed in the sandstone and claystone bedrock near the contact between lithofacies in the Dawson formations.

Hand-held scintillometers have been used to indicate areas of above normal low-energy gamma radiation. The scintillometers provide readings of low-level gamma radiation in terms of micro Roentgens per hour (micro R/Hr). Readings in the Colorado Springs region indicate that typical background radiation readings range from about 15 to 20 micro R/Hr. These levels are above the national average but are typical of the front range.

The Colorado Department of Public Health and Environment (CDPHE) has not established official guidelines regarding the acceptable level of naturally occurring low-energy gamma radiation. However, in conversations with CDPHE personnel levels of concern have been established at about twice the background level. This would imply remediation should be performed for materials that exceed about 30 to 40 micro R/Hr.

Where identified, conglomeratic sandstone float and other soil and bedrock materials that exhibits higher than acceptable radiation levels should be removed. Disposal should be done in accordance with studies showing that a minimum of 2 ft (0.6 m) of clean soil must overlie any low-energy naturally occurring radioactive material (Rogers and Associates Engineering Corporation, 1997). A geotechnical engineering company usually observes this work.

Radon Gas

Radon gas is recognized as a potential hazard in the Colorado Springs area. Radon is a gas that is free to move through the soil and air but can become trapped in structures constructed on the soil. Radon is a by-product of the natural decay of uranium and radium. The majority of the radon gas occurs in areas with elevated levels of low-energy gamma radiation sources in the underlying soils and bedrock. Trace amounts of radioactive nuclides are common in igneous and metamorphic rock and in the soils and sedimentary rocks that underlie this region, so there is a potential for radon gas to accumulate in structures constructed in the region. The primary concern with radon is indoor accumulation radon above the Environmental Protection Agency's guideline levels. The building industry has developed well-accepted mitigation technologies to reduce radon levels.

Flooding and Debris Flow

The Colorado Springs area has experienced flooding since it's early days in the late 1800's. Notable floods occurred in 1921, 1935 , 1965 (Snipes et al, 1974), and 1999. Recognition of flood prone areas is part of the charge of the Geologic Hazard Ordinance of Colorado Springs. Much work has been done to recognize areas prone to impacts from 100-year floods (Federal Emergency Management Agency, 1997). Flooding that is accompanied by debris and mud flows is less widely recognized and requires different and often expensive mitigation compared to water floods (Federal Emergency Management Agency, 2000). Source area geology determines the type and amount of material that is available to form debris flow or be entrained within floodwaters. Geology also impacts topography and gradient, which influence velocity of flows.

Compared to clear-water floods debris flows, mud flows and sediment laden flooding known as hyperconcentrated flooding are classified as non-laminar with viscous properties similar to lava flows in volcanic fields. Non-laminar flow is more difficult to control with engineered works than water flooding because it can contain boulders and other debris that have enormous destructive power.

Perhaps the most notable debris flow events in Colorado Springs area are those associated with the flooding of 1965. The debris flows of 1965 impacted NORAD and destroyed the Ape House at the Cheyenne Mountain Zoo, which was under construction. These debris flows occurred on minor drainages just below the steeper slopes of Cheyenne Mountain and other similar features. The 1965 debris flows occurred after a wildfire had removed most of the vegetation in the source areas in the early1960's. Other more potentially destructive events occurred in this region in recent geologic time, but did not impact human development. Debris flow deposits can be observed from south of Colorado Springs in Cheyenne Mountain State Park, to the drainages below the Rampart Range near the Palmer Divide north of Monument.

Conditions remain favorable for large debris flow events along the margins of the Front Range in the Colorado Springs area. Unusually severe storm cells, associated with summer thunderstorms are the trigger for these type of large and destructive debris flows.

Un-mitigated debris flow hazards exist in the Colorado Springs areas wherever steep mountain drainages exit the Front Range. These hazards may range from hyper-concentrated flooding to boulder entrained debris flows. Recognition of past debris flow and hyper-concentrated flooding events is the first step in determining if an area is at risk from these type of events. Typically, past events have created a debris or alluvial fan where the steep drainage area encounters the change in slope with the valley floor, and debris "trains" consisting of large boulders.

The least expensive mitigation for debris flows is to avoid development of the active debris fan. Mitigation requires that the destructive debris flows be controlled through the use of barriers and debris dams, which are economically viable where the cost can be offset by the value of the land. An example of this trade off is a debris catchment constructed at the mouth of Fisher's Canyon along the base of Cheyenne Mountain. This basin is designed to detain the debris and allow the water to pass for the 100-year design event. This type of structure address the viscous properties of debris flows, along with the highly destructive boulders, trees and other debris entrained the flow. The water that passes through the structure is then managed in conventional drainage systems designed for water floods.

Other similar debris flow structures are being considered in the area. Enlarged channels and structures to allow future debris flows to pass have been designed and constructed in the vicinity of the Cheyenne Mountain Zoo. The Zoo has also developed an emergency evacuation plan to move visitors and staff to safe areas during future events.

A particular characteristic of debris flows is channel switching during flow events. This channel switching is due in part to the viscous nature of the flow, as well as the fact that the large debris sizes can create dams and levees within the moving mass, causing it to switch or jump locations. Hence, the entire alluvial or debris fan can be at risk of future debris flow impacts, not just the present day channel. Mitigation by capturing the debris, typically at the apex of the fan, where the debris flow emerges from the mountain front is often the most desirable way to control the flow. Recognizing the potential for debris flows rather than water flooding is paramount therefore prior to design and construction of site drainage features.

SEISMICITY

Because Colorado Springs is located at the junction of the High Plains and Rocky Mountain topographic provinces, the city straddles the southern end of the Rampart Range Fault and a portion of the Ute Pass Fault. The Rampart Range Fault is characterized as having episodes of normal and reverse faulting by Widmann et al. (2002). The Ute Pass Fault is thought to be either a thrust or high angle reverse fault (Carroll and Crawford, 2000; Widmann et al., 2002).

Colorado Springs, along with most of Colorado, has long been considered an area of low seismicity, with only minor potential for future damaging earthquakes (Algermissien, 1969; Algermissien and Perkins, 1976). Kirkham and Rogers (1981) of the CGS observed that both faults have moved in the Quaternary and the area should be within Zone 2 Unified Building Code (UBC) classification. However, the local building department uses the maps from the 1997 UBC that show the area within Zone 1.

Quaternary seismicity in Colorado is poorly understood. The U.S. Geological Survey seismic hazard mapping of Colorado Springs indicates that accelerations of less than 0.1 g have a 2 percent probability of occurring within the next 50 years (U.S. Geological Survey, 2003). Earthquakes up to magnitude 4.5 have been recorded in the vicinity of the Ute Pass Fault west of Colorado Springs. However, exact locations of the earthquakes have been difficult to assign because of the low number of seismic monitoring stations within Colorado. The Ute Pass Fault primarily offsets Precambrian bedrock, and has created a drag effect in the overlying sedimentary beds, in some cased overturning them (see Niobrara Formation in Carroll and Crawford, 2000). According to Widmann et al (2002) “The best evidence for Quaternary fault activity is limited to the south end of the (Ute Pass) fault system near Cheyenne Mountain where development of a prominent scarp in Verdos Alluvium and scarps extending through Pleistocene rockfall deposits indicate youthful fault activity (Kirkham and Rogers, 1981)." However, not all geologists agree on this evidence and dating of the most recent movements on the Ute Pass Fault is lacking. Varnes and Scott (1973) and Dickson (1986) have reported movement on the Rampart Range Fault based on offset of the Douglas Mesa Gravel (Verdos Alluvium) at the US Air Force Academy.

ENVIRONMENTAL CONCERNS

Water Supply

The first water supplies for the City of Colorado Springs were from wells and diversions from Fountain Creek (Colorado Springs Utilities, 2002). General Palmer, the developer of Colorado Springs, obtained water rights from Fountain Creek in 1870 and constructed the El Paso Canal, which began operation in 1872. The El Paso Canal supplied irrigation water to the downtown and Colorado College areas from 1872 until it's abandonment in 1956. Drinking water in the early years was provided by wells up to 65 ft (20 m) deep in the alluvium adjacent to Fountain and Monument Creeks. Grasshopper infestations in 1874 and 1876 clogged the wells and the El Paso Canal prompting calls for a better water supply.

In 1878 a bond was passed for a new water supply (Colorado Springs Utilities, 2002). In 1891 the south slope water system was purchased, which included seven reservoirs on the south slope of Pikes Peak and a dam was added to Moraine Lake. The dams in the system were improved over time: Boehmer in 1894, Bighorn and Wilson in 1896, Mason and McReynolds in 1905, and Big Tooth in 1929. The system included seven tunnels, the Ruxton and Manitou Hydroelectric power plants, and was serviced by the Mesa Water Treatment Plant. The south slope system supplied adequate water until the 1930's, when demand exceeded its capability to supply.

Work began on the north slope water supply system with survey work in 1901, and acquisition of water rights on the north slope of Pikes Peak between 1908 and 1930 (Colorado Springs Utilities, 2002). During the 1930's Crystal and South Catamount Reservoirs were constructed as WPA projects. The system was then able to meet demands until after World War II and the rapid growth of the city in the 1950's. In the 1950's the first trans-mountain diversions for water for the City of Colorado Springs occurred from the upper Blue River south of Breckenridge. This required 70 mi (112 km) of pipeline, and the water was stored in North Catamount Reservoir on the north slope of Pikes Peak. In the 1960's the system added 10 tunnels that supplied water through the Ute Pass and Mesa Water Treatment Plants. Homestake I reservoir and tunnel was completed in 1967 as a joint project with the City of Aurora and included a 5.6 mi (9 km) tunnel under the continental divide to an enlarged Turquoise lake, on the Lake Fork of the Arkansas River. By the late 1970's the Fry-Ark Project was on line and supplying additional trans-mountain diversion water from the upper reaches of the Fryingpan and Roaring Fork Rivers via a series of tunnels including the South Fork, Charles Boustead and Homestake Tunnels to Turquoise Lake. Water from Turquoise Lake is currently piped to Twin Lakes or pumped to the Mount Elbert Forebay. Water flow from the Mount Elbert Forebay to Twin Lakes is used to generate peak power. Water released from Twin Lakes and flows down the Arkansas River to

Pueblo Reservoir. Water is pumped from the Otero Pumping Station on Pueblo Reservoir via 50 mi (79 km) of 66-inch-diameter (1.7 m) and 26 mi (42 km) of 48-inch-diameter (1.2 m) concrete pipe to six water treatment plants around the city. Some water from Twin Lakes is diverted via pipelines to Rampart Reservoir northwest of Colorado Springs (Colorado Springs Utilities, 2002).

Currently Colorado Springs water is supplied primarily by surface water flows with 18 percent of the water coming from the slopes of Pikes Peak and more than 75 percent from trans-mountain diversions through the Fry-Ark project and the Blue River diversions (Colorado Springs Utilities, 2002).

Because of continuing population growth and the drought in 2001-2003 the city-owned utilities were forced to begin mandatory watering restrictions. The watering restrictions began in summer of 2002 with outdoor watering limited to three hours per day, two days per week. In the fall of 2002 watering was further reduced to two days per month. In the longer term, the city is looking into adding more storage capacity to its system and constructing a new raw water line from Pueblo Reservoir to a water treatment plant on the south side of the city.

Manitou Springs obtains its water from runoff from Pikes Peak and from wells. The majority of the water supply in the Monument area is from wells in the Dawson and Laramie Formations. Fountain obtains about 70 percent of its water from the Fry-Ark project via Pueblo Reservoir and about 30 percent via shallow wells tributary to Fountain Creek

Wastewater Disposal

Flows from the City of Colorado Springs Utilities sanitary sewer system are currently treated at the Las Vegas Street treatment plant (CSU, 2002). The plant includes primary, secondary and tertiary treatment. Sludge from the secondary treatment is disposed of via an 18- mi-long (29 km) pipeline to Clear Springs Ranch. After tertiary treatment a portion of the water is being reused for non-potable irrigation water. This has required the construction of a separate non-potable water system. The non-potable reuse of water is limited to trans-mountain diversion water only, since east slope water is limited to a single use, by Colorado Water Law. Similar non-potable reuse is occurring in other Colorado Front Range cities with trans-mountain diversion water. Smaller municipal waste water treatment facilities are located in Fountain, Manitou Springs, and the Monument area.

Hazardous and Solid Waste Disposal

There are four sanitary landfills operating locally that are open to the public. They are located east, southeast, and south of Colorado Springs. There are numerous small closed landfills scattered around the city, most of which were only operational for short periods of time. The Galley Road Dump Site, Hancock Plaza Landfill, Institute Dump, Pinello Landfill, and Templeton Gap Landfill are all former landfills that are listed as waste sites by the Colorado Department of Public and Environmental Health (Colorado Department of Public and Environmental Health 2003). There were also numerous unregulated dump sites along many of the drainages through the city. In addition to the dumps listed above there are 26 other hazardous waste sites in El Paso County (Colorado Department of Public and Environmental Health, 2003). About one-third of the sites are related to heavy metals either from production at mills or use in manufacturing (Colorado Department of Public and Environmental Health, 2003).

MAJOR ENGINEERED STRUCTURES

The major engineered structures in the Colorado Springs area are limited to the few midrise buildings (6 to 20 stories) in the downtown area, hospitals or at the Broadmoor Hotel Complex; public facilities - World Ice Area, Garden of the Gods Visitors Center, US Olympic Training

Table 1. Major Engineering Structures in Colorado Springs

Structure	Date of Construction	Foundation Type	Geologic Supporting Units	Comments
Broadmoor Hotel Main Building	1916-18	Spread Footings	Clay, sand and gravel of the Verdos Alluvium	
NORAD	1964-65	Steel Springs	Cheyenne Mountain granodiorite	Buildings and water storage tanks built in tunnels
Antlers Hotel and Holly Sugar	1965-67	Drilled piers through alluvium to bedrock	Alluvium with groundwater over Pierre Shale	
World Ice Arena	1996	Drilled Piers	Alluvium over Pierre Shale	
Garden of the Gods Visitors Center	1994-95	Footings on 10 ft of granular fill	Steeply dipping expansive Pierre Shale	Fill drains to gravity outlet on site
Memorial Hospital expansion	1996	Drilled Piers	Eolian Sand over Pierre Shale	
Penrose Hospital Expansion		Tower – Drilled Piers Other portions – Spread Footings	Louviers Alluvium over Pierre Shale	
US Olympic Training Center	1993-97	Footings on Fill	Hydro-compactive Eolian Sand over Pierre Shale	Site transferred to USOC form North American Defense Command in 1978
Airport Terminal	1993-94		Eolian Sand, and Broadway Alluvium over Pierre Shale	

Center and the Airport terminal; or federal facilities like NORAD. Some of the major engineering structures in the Colorado Springs area are listed in Table 1.

USE OF UNDERGROUND SPACE

The significant uses of underground space in the Colorado Springs area are the Cheyenne Mountain Air Force Station underground operations center, known as NORAD, and water supply tunnels with hydro power stations. The NORAD facility was excavated into the base of Cheyenne Mountain to provide a hardened command center for potential counter attacks using nuclear missiles. The excavation for NORAD was begun in June 1961 and the facility was operational on January 1, 1966. The facility has been in use since then and continues to serve as a space tracking center (U.S. Air Force, 2003A).

The facility has two entry tunnels that were excavated by drill and blast methods into the Pikes Peak granite (U.S. Air Force, 2003B). The facility inside the mountain covers approximately 4.5 acres (1.8 hectares), contains 15 free-standing buildings and four 1.5 million gallon (5.67×10^6 *l*) reservoirs. The excavation removed approximately 700,000 tons (635,000 Mg) of granite. There are about 110,000 rock bolts from 6 to 32 ft (1.8 to 9.8 m) long reinforcing the tunnel structure.

The primary local water supply tunnels are the Rampart Range Tunnel and hydro power station and the tunnel to the Tesla hydro power station.

The only other common use of underground space in Colorado Springs is basements beneath some of the large buildings downtown and many of the residences in the area.

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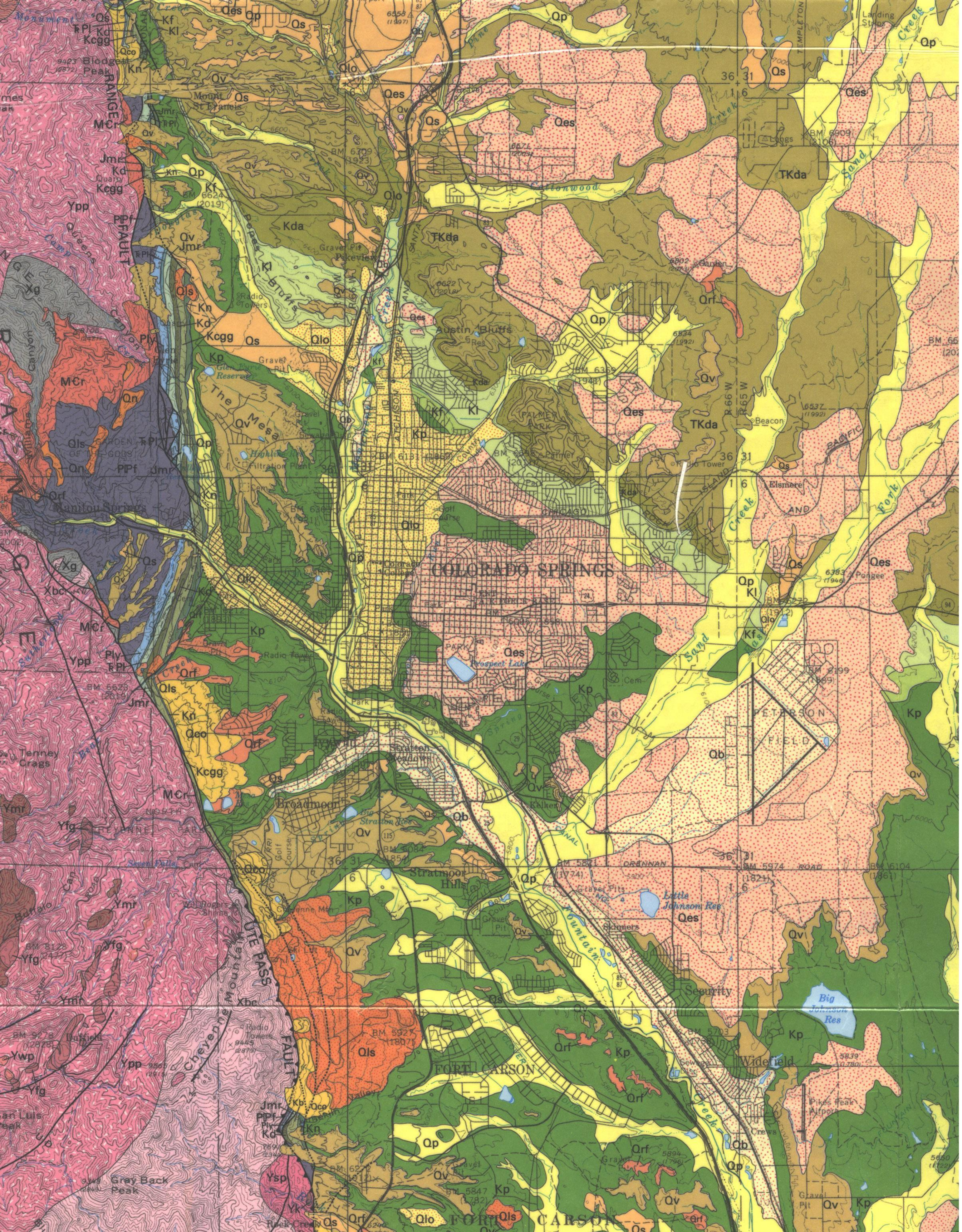
MISCELLANEOUS INVESTIGATIONS SERIES
COLORADO SPRINGS—CASTLE ROCK AREA, COLO.
MAP I–857–F



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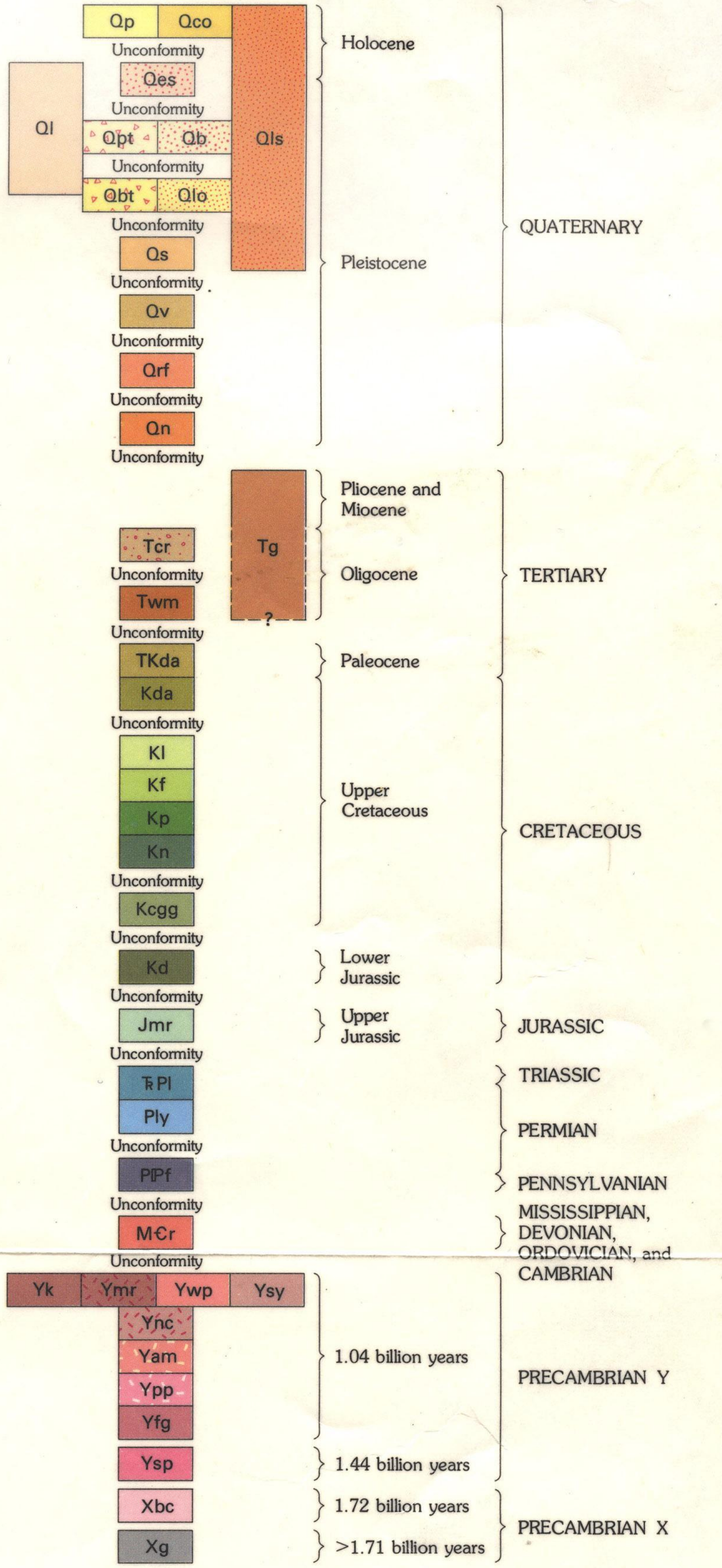

HISTORY OF ENGINEERING GEOLOGY DEVELOPMENT IN COLORADO

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***Key Terms**: engineering geology, history, Colorado, dams, tunnels, highways, landslides, rockfalls, debris flows, floods, earthquakes*

ABSTRACT

The history of engineering geology practice in Colorado is replete with examples of significant innovations and contributions by individuals and organizations. Activities in Colorado that can be classified as belonging to engineering geology can be traced back to the early 1900's, long before engineering geology was generally recognized as a distinct discipline. Over the years, as Colorado faced numerous challenges in designing and developing the infrastructure to support expanding population and economic activities, engineering geology became recognized as important to longer range planning as well as to the success of many projects of all sizes.

Many factors have combined to support and direct the evolution of engineering geology in Colorado. In the period since the 1970's, increased emphasis on environmental values has materially changed the role and scope of engineering geology. Interactions among various levels of government, private firms, and academic institutions further expanded and diversified engineering geology activities. Professional meetings and shared projects encouraged technology transfer that led to many creative advances and contributions to the practice of engineering geology.

This paper first summarizes the population growth and climate issues of Colorado, then traces the engineering geological developments in Colorado in light of human demands for water supply, transportation, growth of population into hazardous areas, and responses to societal concerns. Space prevents a complete listing of all worthy applications of engineering geology within Colorado; the examples used include those considered to be landmarks by the authors.

INTRODUCTION

Colorado has a distinguished engineering geology history and has been home to many engineering geologists. Activities in Colorado that can be classified as belonging to engineering geology can be traced back to the early 1900's, long before engineering geology was generally recognized as a distinct discipline. These activities reflect surges in population and economic activities within the state. Initial settlement was dependent upon gold and silver mining, causing many people to settle in hazardous mountain locations. Precedent-setting reports, such as the 1909 U.S. Geological Survey Professional Paper dealing with landslides in the San Juan Mountains (Howe, 1909), resulted from early studies in response to these hazards. Howe's report not only described landslides, it provided an analysis of their causes and a first attempt at landslide classification. Changes in technology and societal values have impacted the practice of engineering geology. For example, the increased emphasis on environmental values in the period since the 1970's has materially changed the role and scope of engineering geology in Colorado.

Four characteristics combine to make Colorado a natural focus for engineering geology:

- Rapid population growth
- Semi-arid climate with marked wet and dry cycles
- Distinct topographic regions
- Varied and complex geologic setting

The first two characteristics represent "extrinsic," or non-geological, factors. Rapid population growth is the socio-economic driving force that creates demands for infrastructure elements that provide shelter, food, and transportation. Colorado's semi-arid climate with its marked wet and dry cycles further increases the engineering geological challenges; both from a hazards perspective and also because complex and costly dams, tunnels, and canals are required to meet water-supply needs.

The third and fourth characteristics may be considered as "intrinsic," or geological and engineering, factors. The Colorado landscape with its distinctive regions of Eastern Plains, Central Mountains, and Western Plateaus, combined with the varied and complex geologic conditions and resources within each, encourage settlement of people in all parts of Colorado. The topography and geologic materials present a large variety of engineering geological challenges to providing the necessary infrastructural elements.

Thus, this paper first summarizes the population growth and climate issues of Colorado, then traces the engineering geological developments in Colorado in light of human demands for water supply, transportation, growth of population into hazardous areas, and responses to societal concerns.

COLORADO POPULATION GROWTH AND DISTRIBUTION

The 2000 Census documented that Colorado has a population of 4.3 million (Table 1). These data also show a marked acceleration in population growth over time. Assuming the first settlers arrived in the 1850's, it took 80 years (1850-1930) for Colorado to reach 1-million residents. After another 40 years (1930-1970) the population reached 2-million, then it took only 20 years (1970-1990) to reach 3-million, and only 10 years (1990-2000) to reach 4-million! This accelerating growth results in growing demands for shelter, infrastructure (transportation and water supplies), and for construction materials and environmental controls, including waste disposal. Because these issues can often only be resolved by engineering geology investigations, it should not be surprising that Colorado has been the focus of many advances in engineering geology.

Table 1. Total Colorado Resident Population (Source: U. S. Bureau of Census data)

Year	Resident Population
1930	1,035,791
1940	1,123,296
1950	1,325,089
1960	1,753,947
1970	2,209,596
1980	2,889,735
1990	3,294,473
2000	4,301,261

Of equal concern is that this population growth has not been equally distributed throughout the state, rather it has been concentrated in 10 counties along the Colorado Front Range (Figure 1). Figure 2 shows this trend clearly, and includes Census projections to the year 2020. However, not all counties within this group of ten are similarly affected; some, such as Denver, are mature urban centers with relatively stable populations since the 1960's, while others, such as Douglas County are experiencing some of the highest population growth rates in the USA (Figure 3). These statistics combine with other factors to provide important engineering geological concerns. Two concerns that will be described in greater detail in following sections are the potential difficulties in ensuring adequate long-term water supplies to the growing Front Range population, and the expansion of populations into geologically hazardous environments.

COLORADO'S SEMI-ARID CLIMATE

Colorado enjoys a relatively mild semi-arid climate. Average annual precipitation in Denver is about 13 in. Potential evapotranspiration is much greater – some 3 to 4 times the average precipitation. These values change with elevation. At higher elevations in the mountains the precipitation increases, so that at 10,000 ft, 30 in is common, and evapotranspiration is much lower. Dominant precipitation comes from the west, thus those portions of Colorado lying to the west of the Continental Divide get more precipitation than the eastern parts of the state. For example, the South Platte River carries less than 10% of the average annual streamflow in the state, while more than 70% of the state's surface water flows westward in the Colorado River basin.

Although the dominant surface water supplies are found on the western slope, the bulk of Colorado's population is found in the east, along the Front Range (see previous section). To make matters even more difficult, precipitation is not evenly distributed throughout the year. Much precipitation falls as snow in the mountains during the winter and early spring, so streams reach a peak flow due to snowmelt in late spring and early summer. Flows are much lower at

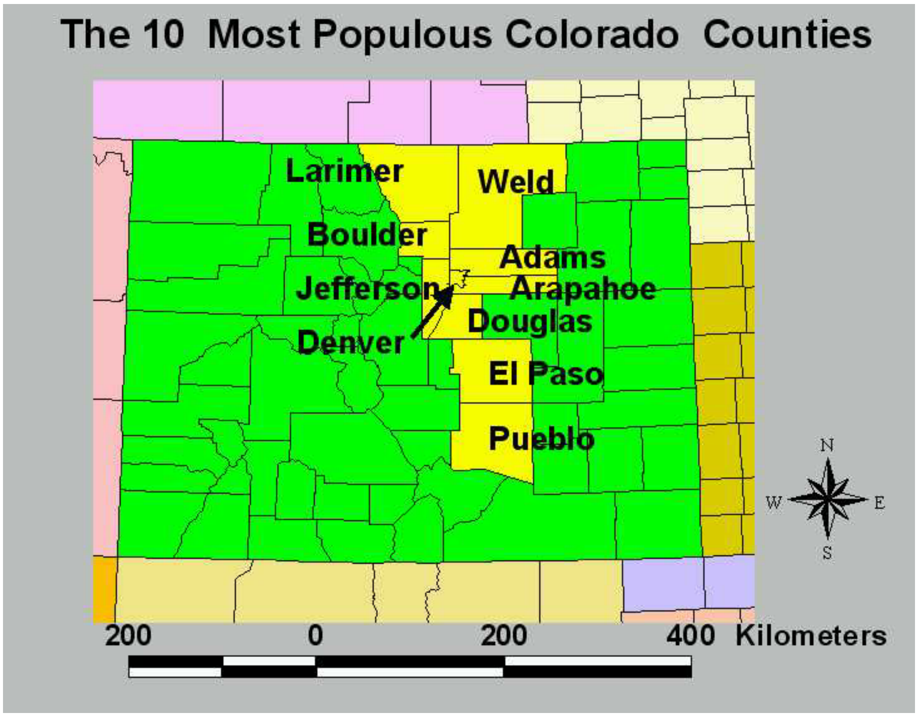

Figure 1. The ten most populous Colorado counties (U.S. Bureau of Census data).

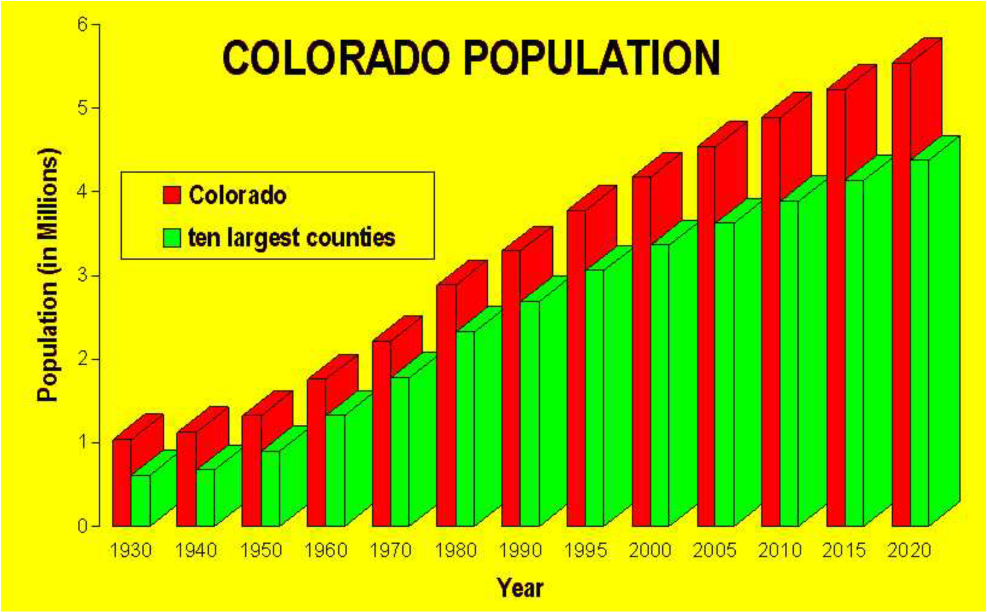

Figure 2. Comparison of Colorado population growth state-wide and in the ten most populous counties (U.S. Bureau of Census data).

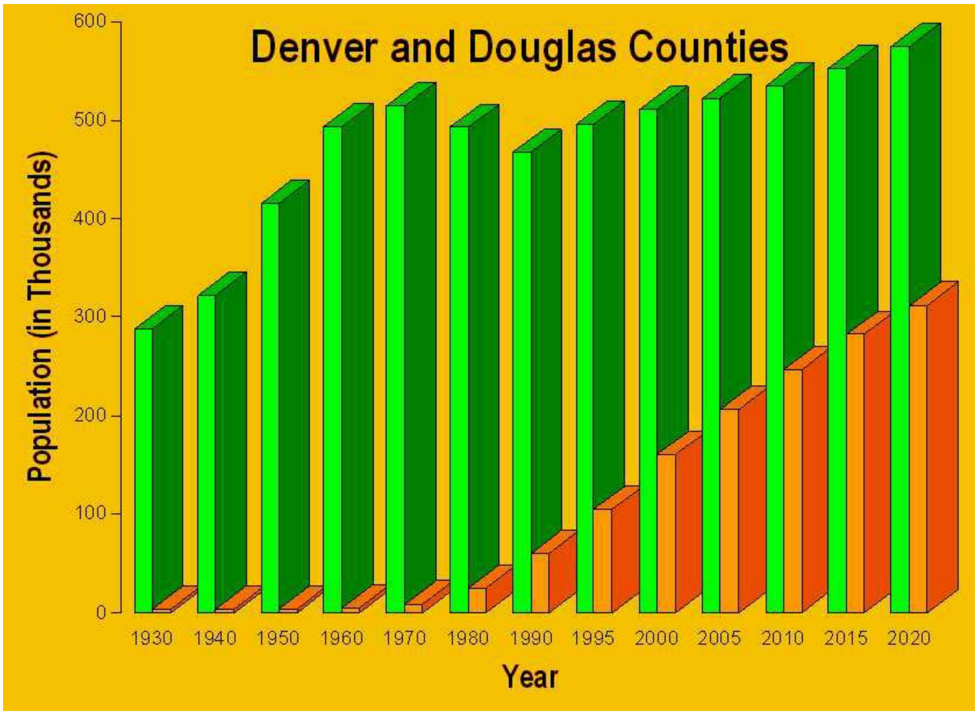

Figure 3. Comparison of population growth in Denver and Douglas counties (U.S. Bureau of Census data).

other times of the year. Human demands for water tend to peak in the summer, due to needs for irrigation. Figure 4 illustrates this situation using data from Cherry Creek near Denver.

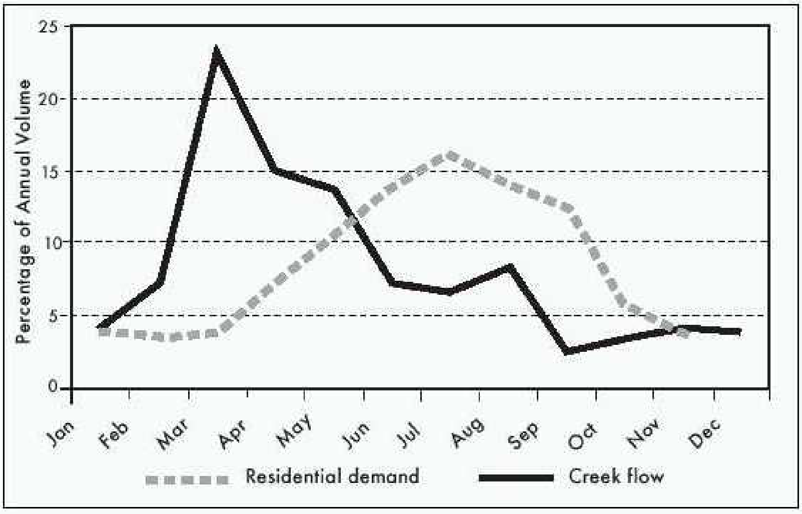

Figure 4. Plots of normalized stream-flow in Cherry Creek and residential demand show the out-of-phase timing of surface water supply with demand (Colorado Geological Survey, 2002b - data courtesy Parker Water and Sanitation District).

The mismatch between the timing of peak supply and peak demand demonstrates the need for construction of water storage and management schemes. The spatial separation of more plentiful surface water supplies west of the Continental Divide and population centers to the east of it, led to demands for complex and expensive systems to divert western water supplies across the Continental Divide to eastern markets. These “transmountain diversions” represent large engineering geological challenges. They also are the subjects of major political, economic, and environmental debates.

In 2002 Colorado experienced the driest twelve-months on record in precipitation records that extend back about 150 years (Welsh, 2003). The drought has severely impacted many aspects of Colorado’s economy, especially because the population is much larger than in previous dry periods. Modern society contains economic sectors, such as recreation and tourism, which contribute significantly to the Colorado economy and are adversely impacted by drought (Schneckenburter and Aukerman, 2003).

Archeological and historical research suggests that the entire southwestern USA is subject to alternating wet and dry cycles. Significantly, there appears to be a long-period cycle lasting around 400 years, and superimposed on this cycle are shorter 20- to 25-year wet and dry cycles (Welsh, 2003). Archeological evidence suggests that the ancient Native American cultures were attracted to the “Four Corners” region during a 400-year wet cycle lasting between roughly 800AD and 1200AD. Their civilizations became increasingly stressed in the subsequent dry cycle, so that they constructed cliff-dwellings to provide defenses against attacks and subsequently migrated by the mid-1400’s. The next wet cycle beginning in the 1600’s coincided with the arrival of Spanish explorers. Thus the entire 400-year period of European and American control of Colorado has been during a period of comparative water abundance – tempered with a series of 20- to 25-year wet and dry cycles (Welsh, 2003). This analysis suggests that the current drought may possibly indicate the start of a long period of water shortages.

The pulses of historical settlement of Colorado often fortuitously coincided with the shorter 20- to 25-year wet cycles. For example, the period 1865-1885 was a wet cycle and during this period the early agricultural developments on the Eastern Plains, such as those around Greeley, flourished with the use of local irrigation diversion canals capturing the local eastward flowing streams from the Rockies. The following period from 1885 to 1905 was comparatively dry. This caused many of the privately financed local irrigation systems to fail to provide sufficient water. This period also corresponded to the “Silver Panic” of the 1890’s and the collapse of much of Colorado’s mining economy. The economic distress caused many to move out of the State, somewhat alleviating the shortage of local foodstuffs.

Those that remained began to agitate for Federal governmental help to secure adequate water supplies to maintain agriculture. The Federal Reclamation Act of 1902 established the Reclamation Service (the fore-runner of the Bureau of Reclamation) under the jurisdiction of the U.S. Geological Survey. It became an independent bureau within the U.S. Department of Interior in 1907. The Reclamation Act authorized the Secretary of Interior to locate and construct irrigation works in 16 western states and territories. The works were funded by sale of public lands. Projects began immediately, including several in Colorado. The Bureau of Reclamation, with its major Engineering and Research Center (now named Technical Service Center) located at the Denver Federal Center in 1943, has been an important component of engineering geology activity in Colorado. Its major Colorado projects are described subsequently.

Denver enjoys senior appropriation rights to many surface water supplies, and has developed an extensive water supply infrastructure to capture and manage them, including transmountain water diversions. Such sources are not available to other later developing and rapidly growing areas, such as Douglas County. Growth there has relied on ground water resources and these are being rapidly depleted. Other rapidly growing communities, such as Colorado Springs, Aurora, and Fort Collins, have access to somewhat less extensive transmountain diversion systems, similar to and to some degree in competition with Denver's system. They are supplementing these supplies with ground water withdrawals. Although each community has different levels of difficulty in meeting their current and future water-supply requirements, there is an increasing awareness of pending water supply shortages.

ENGINEERING GEOLOGY AND WATER SUPPLY

Water supply has been a key factor in development in Colorado from the earliest settlements to the present. Population growth, combined with the semi-arid climate and wet and dry cycles, creates ever-larger demands for water. The earliest water-supply schemes involved relatively small-scale diversion canals and relatively low dams. The comparatively dry period from 1885 to 1905 saw the construction of several considerably larger dams, including Castlewood Dam on Cherry Creek in 1890, and Cheesman Dam on the South Platte in 1905. These early works were built with little geological input, and some failures were the result (see Castlewood Dam below). Establishment of the Bureau of Reclamation and the Denver Water Board in the early 1900's began a period of remarkable construction of dams, tunnels, and aqueducts for water supply, storage, and regulation throughout Colorado.

With considerable and ever increasing engineering geological input, transmountain water storage and diversion schemes were designed to make the more abundant western slope (Colorado River Basin) water supplies available to the growing eastern slope populations. The Bureau of Reclamation developed the Colorado-Big Thompson Project in northern Colorado between 1938 and 1957, and the Fryingpan-Arkansas Project in Southern Colorado between 1964 and the mid-1980's. On the western slope, during the 1970's, the Bureau of Reclamation constructed a series of dams on the Gunnison River (the Aspinall Unit) to provide water storage and management under terms of the interstate water sharing procedures mandated by the Colorado River Compact.

The Denver Water Board, the City of Aurora, and the City of Colorado Springs continued to construct a series of increasingly elaborate diversion schemes until environmental concerns led the U.S. Environmental Protection Agency in 1996 to disallow the construction of Two Forks Reservoir by the Denver Water Board on the South Platte River in the foothills southwest of Denver. This action effectively terminated construction of large-scale water storage schemes in Colorado.

The U.S. Army Corps of Engineers constructed three large flood-control in reservoirs in the Denver area – Cherry Creek Reservoir, completed in 1950, Chatfield Reservoir on the South Platte, completed in 1976, and Bear Creek Reservoir west of Denver, completed in 1976 (Costa and Bilodeau, 1982).

Numerous engineering geologists were involved in these many projects. Some were employees of the federal, state, and local government entities named above who were responsible for the design, construction, and operation of the various systems. Others were employed as private consultants to provide expertise on specific aspects. The engineering geologists faced a variety of challenges. Large dams of many designs were constructed, as well as several very long hard-rock tunnels, and many miles of canals, siphons, and aqueducts.

Dam Failures

It is a measure of the skill of the engineers and geologists involved in these projects that most have performed well over many years. On the later (and larger and more complex) projects, failures have been rare and minor. However some of the early smaller projects, constructed without the benefit of geological consultation, have experienced failures.

Castlewood Dam: One of the earliest notable dam failures in Colorado was the 1933 failure of Castlewood Dam, located on Cherry Creek south of Denver (Costa, 1978, pp.24-27). Today the remnants of the dam form a focal point for one of Colorado's State parks (Figure 5).

Figure 5. Castlewood Dam Today.

In 1890, Castlewood Dam was built across Cheery Creek about 40 mi south of Denver to provide irrigation to Mennonite cherry orchards south of Denver. The dam was a combined masonry and rockfill structure 600 ft long and with a maximum height of 80 ft. The dam was located in a steep gorge eroded by Cherry Creek into the Castle Rock Conglomerate, Wall Mountain Tuff (Douglas Rhyolite), and the Dawson Formation, a poorly cemented, very friable arkosic sandstone with interbedded claystone. Differential settlement of this poor foundation caused the dam to crack and leak soon after construction. An "impermeable" earthen blanket was placed in the reservoir upstream of the dam to reduce leakage.

The dam was repeatedly declared unsafe by a series of State Engineers, but remained in operation until the night of August 2, 1933, when an intense rainstorm provided up to 8-in of rainfall over the headwaters of Cherry Creek. The dam was overtopped and the right section collapsed in the early morning of August 3rd. A peak discharge estimated at 126,000 cubic ft per second (cfs) flowed down the gorge and into the wider valley of lower Cherry Creek. Valley storage fortunately reduced the peak flow to about 16,000 cfs by the time the flood crest reached Denver some six-hours later. The flood stage was only about 7 in above the walled concrete channel along today's Speer Boulevard through Denver, but damage was still estimated at $800,000 (Costa, 1978).

This flood led to the construction of the Kenwood Dam on Cherry Creek in southeastern Denver between 1935 and 1936 (Costa, 1978; Costa and Bilodeau, 1982). However a major storm that occurred in 1935 demonstrated that the Kenwood Dam was under-designed and it was considered obsolete even before it was completed (Costa, 1978). Kenwood Dam was replaced by the much larger Cherry Creek Dam in 1950, as part of the U.S. Corps of Engineers flood protection system for Denver.

Lawn Lake Dam: On July 15, 1982, a 26-ft-high earthen dam located in Rocky Mountain National Park upstream from the Town of Estes Park, failed at the height of the summer holiday season. Originally constructed in 1903 to supply irrigation water, the dam was not properly maintained. Subsequent investigations determined that leaks around an outlet pipe on the upstream side of the dam eroded the earthfill, and progressive deterioration led to failure of the embankment.

The failure released an estimated 674 acre-ft of water with a peak discharge rate of 18,000 cfs down the Roaring River valley. The flood killed three campers. Surviving campers along the Roaring River estimated a wall of water 25-30 ft high came down the valley.

Some 6.7 mi downstream a second dam was over topped and failed. Cascade Lake Dam was a 17-ft high concrete gravity dam that retained an additional 12.1 acre-ft of water. The renewed flood swept down the Fall River into downtown Estes Park where normal summer flows are about 100 cfs. Extensive property damages, totaling $31 million, resulted (Figure 6). Estes Park subsequently undertook a series of urban renewal and flood hazard mitigation projects to reduce future flood hazards.

Denver Water Supply System

Denver supports the biggest and oldest transmountain diversion system in Colorado that brings water from western slope to eastern plains (Denver Water Board, undated; Costa and Bilodeau, 1982). The system began in 1890's as private company, but became a public utility, the Denver Board of Water Commissioners, in 1918. Over the years the system has greatly expanded and currently includes nine large reservoirs, four major tunnels (longest is 23.3 mi), and five major water treatment facilities. The system consists of two major parts, named the "South Platte River System" and the "Northern Collection System." (Table 2). The South Platte River System contains most of the older "eastern slope" facilities, but was considerably enlarged with

Figure 6. Flooding in the Town of Estes Park following Lawn Lake Dam Failure (Photo courtesy Town of Estes Park).

transmountain diversions in the mid-1960's (Dillon Dam/Reservoir and Roberts Tunnel). The Northern Collection System dates from the mid-1930's when the first transmountain diversions through the Moffatt Water Tunnel became feasible. The Denver system also supplies water to many customers in suburban communities outside the City of Denver.

The Denver water supply system has developed with the assistance of many notable engineering geologists; some of their accomplishments are described in the following sections.

Cheesman Dam: The oldest dam in the Denver Water Board water supply system, Cheesman Dam was completed in 1905 by a precursor private company and was purchased by the Denver Water Board in November 1918. The dam is a gravity arch masonry dam, the first of its type in the USA and was the world's tallest dam at 221 ft above the streambed when completed in 1905 (Figure 7). The dam has been designated a National historic Civil Engineering Landmark. Cheesman remains an important part of Denver's mountain storage facilities with a reservoir that holds nearly 80,000-acre ft of water. (Figure 8).

Moffat Water Tunnel: The construction of the Moffat railroad tunnel provided a pilot bore, that was acquired by the Denver Water Board in 1927. The pilot bore was subsequently enlarged and partially lined and placed in service in 1936 to provide the first transmission of water eastward under the Continental Divide. The tunnel was fully lined in 1958. It conveys up to 1,280 cfs of water from the Williams Fork and Fraser River collection systems into South Boulder Creek and Gross Reservoir, thence to Ralston Reservoir and the Moffat Treatment Plant.

Harold D. Roberts Tunnel: Completed in 1962, after 16 years of construction, the Roberts Tunnel is the world's longest major water tunnel – 23.3 mi (Figure 9). It conveys water from Dillon Reservoir eastward into the North Fork of the South Platte River. The construction faced many problems, being as deep as 4,465 ft below the surface. Important problems included the

Table 2. Principal Components of the Denver Water Supply System
(treatment and distribution plants not shown)
(SOURCE: Denver Water Board, undated)

System/Component	Year Constructed	Characteristics
SOUTH PLATTE RIVER SYSTEM		
City Ditch	1867	27.5-mi irrigation supply
High Line Canal	1883	68-mi irrigation canal
Marston Reservoir/Dam	1902	Reservoir for Marston Treatment Plant
Platte Canyon Reservoir	1904	Reservoir for former Kassler Treatment Plant
Cheesman Dam/Reservor	1905	masonry, gravity arch
Antero Reservoir Dam/Reservoir	1909	earth fill dam
Eleven Mi Canyon Dam/Reservoir	1932	concrete gravity arch dam
Harold D. Roberts Tunnel	1962	23.3 mi-long, 10.25-ft diameter fully lined tunnel
Dillion Dam/Reservoir	1963	earth fill dam
Marston Intake Dam	1964	concrete diversion dam
Strontia Springs Dam/Reservoir	1983	Double curvature thin arch concrete dam
NORTHERN COLLECTION SYSTEM		
Moffat Water Tunnel	1936	6.1-mi, 10.5-ft diameter fully concrete-lined tunnel
South Boulder Diversion Intake Dam	1936	gravity concrete dam
Fraser River Diversion System	1936 +	27.7-mis open & closed conduits and canals
Ralston Dam/Reservoir	1937	earth fill dam
Williams Fork Dam/Powerplant/Reservoir	1938 Enlarged 1959	thin arch concrete dam with powerplant
Williams Fork Diversion System	1940	3.9-mi closed conduit
August P. Gumlick Tunnel	1940	2.9-mi, 7-ft horse-shoe-shaped, concrete tunnel
Gross Dam/Reservoir	1954	gravity-arch concrete dam
Vasquez Tunnel	1958	3.4-mi, 7-ft horse-shoe-shaped, concrete tunnel

swelling of crushed and altered rock associated with faults (Wahlstrom et al., 1966). These problems were similar to those encountered during the earlier construction of the Moffat Tunnel.


Figure 7. Construction of Cheesman Dam (photo courtesy Denver Water Board).


Figure 8. Aerial view of Cheesman Dam (photo courtesy Denver Water Board).

Figure 9. View of Roberts Tunnel under construction (photo courtesy Denver Water Board).
and so should have been anticipated. The tunnel was advanced on four headings, from each portal and from two shafts located 5.4 and 8.7 mi from the west portal.

Dillon Dam and Reservoir: Dillon Dam is a large earth-fill dam completed in 1963, with a maximum height of 231 ft, and a crest length of 5,888 ft. It has a crest width of 32 ft, and a maximum base width of 1,100 ft. The dam contains more than 12 million cubic yards of fill (Figure 10).

Figure 10. Dillon Dam nearing completion (photo courtesy of Denver Water Board).

The dam is founded on faulted and extensively jointed Mesozoic sedimentary rocks, and on pervious gravels up to 80 ft deep in the valley below the Blue River. (Wahlstrom, 1966) Geologic features were defined by preliminary exploratory drilling and detailed studies during construction and allowed close control of foundation excavation and grouting. Special problems arose from heavy groundwater flows from the pervious gravels and from repeated movements of an ancient landslide that was reactivated by loading from a temporary earth embankment and simultaneous excavation near its toe (Wahlstrom, 1966). To accommodate the reservoir, it was necessary to relocate the entire town of Dillon, 13 mi of highway, 8 mi of transmission line, a hydroelectric generating plant, and other facilities. Dillon Reservoir nearly doubled Denver's water storage capacity. It is one of Colorado's largest bodies of water and a major recreational facility.

Strontia Springs Dam: In order to provide additional eastern slope storage and thus allow for peak demands in excess of the capacity of the Roberts Tunnel, a new reservoir was constructed in the narrow Platte River Canyon southwest of Denver (Figure 11). The site was suitable for a comparatively thin and high (234 ft) double curvature thin arch concrete dam, which was completed in 1983. This necessitated careful engineering geological investigations of the abutments. Additional engineering geological investigations were required to evaluate the potential for landslides or rockslides into the reservoir in order to ensure a safe facility.

Figure 11. Strontia Springs Dam under construction (photo courtesy Denver Water Board).

Two Forks Dam: Perhaps the most famous dam in the Denver water supply system is the one that was ultimately denied a permit for environmental reasons – Two Forks Dam. The proposed dam location is approximately one mi downstream from the confluence of the North Fork of the South Platte River and the mainstream of the South Platte. The dam would have stood 615 ft high and spanned some 1700 ft. The reservoir created by the dam would have had a surface area of 11.4 square mi, and flooded approximately 30 mi of the river.

Over a period of many years the U.S. Environmental Protection Agency opposed the dam on environmental grounds. Arguments against the dam included the loss of "one of the most highly prized and used trout fisheries in the nation" and impacts on whooping crane habitat hundreds of mis away along the South Platte River in Nebraska. On June 5, 1996, a federal judge upheld the EPA decision to block the dam's construction and reaffirmed the Justice Department's enforcement of the Clean Water Act. This effectively marked the end of large dam construction in Colorado (U.S Department of Justice, 1996).

U. S. Bureau of Reclamation Activities

The U. S. Bureau of reclamation traces its origins to the Federal Reclamation Act of 1902, which established the U.S. Reclamation Service under the jurisdiction of the U.S. Geological Survey. A reorganization in 1907 converted the Reclamation Service to an independent Bureau of Reclamation within the U. S. Department of Interior. Projects began immediately and the Bureau of Reclamation soon established itself as a world leader in dam engineering. Many of its projects were located outside Colorado. However a reorganization in 1943 brought considerable numbers of Bureau engineers and geologists to Denver to staff the Engineering and Research Center (now named Technical Service Center) located at the Denver Federal Center. This has been an important component of engineering geology activity in Colorado (Simonds, 1998).

From its beginnings to the present day, the Bureau of Reclamation has undertaken a number of projects in Colorado with significant engineering geology aspects. Five of these projects are described in the following sections. The Uncompahgre Project was among the first five projects authorized and included construction of the longest irrigation water tunnel in the world at that time. The Colorado-Big Thompson Project was built between 1938 and 1957, and is the largest transmountain water diversion project in Colorado. The subsequent Fryingpan-Arkansas Project provides a similar transmountain water diversion for south-central Colorado. A series of three hydro-electric dams and storage reservoirs constructed on the Gunnison River of western Colorado between 1962 and 1976 form the Aspinall Unit of the Bureau's Colorado River Storage Project. The recent modernization of the Horsetooth Reservoir dams near Fort Collins is an example of the current activities undertaken by the Bureau.

The Uncompahgre Project and Gunnison Tunnel: On March 14, 1903, following passage of the Reclamation Act in 1902, the Secretary of the Interior authorized the Uncompahgre Project (originally called the Gunnison Project), as one of the first five projects of the United States Reclamation Service (Clark and Simonds, 1994). Located just east of Montrose, Colorado, the Uncompahgre Project diverts water from the Gunnison River in the Black Canyon to provide irrigation in the Uncompahre Valley around Montrose and Delta. The Uncompahgre Project includes the Gunnison Tunnel, one storage dam, several diversion dams, 128 mi of canals, 438 mi of laterals and 216 mi of drains to irrigate 66,000 acres on the western slope of Colorado at an elevation between 5,000 and 6,000 ft above sea level.

In the 1880s, when it became apparent that the Uncompahgre Valley was capable of producing crops, especially fruit, a great race for land took place, and a number of local irrigation enterprises began to use water from the Uncompahgre River and its tributaries. However it soon became clear that these water supplies were insufficient and by 1890 water shortages were common. Demands for more water increased during several dry seasons, so that by 1901 the State legislature authorized $25,000 to construct a 3 mi long tunnel allowing diversion of water from the Gunnison River to the Uncompahgre Valley. By the fall of 1902 this project was abandoned due to lack of funds. Work was restarted in 1905, following geologic and topographic surveys by the U. S. Geological Survey, plus investigations carried out by the

Reclamation Service. A new tunnel alignment, some 5 mi upstream from the original tunnel, was selected as being more feasible and appropriate (Clark and Simonds, 1994).

Construction of the new tunnel experienced great difficulties – road access into the Gunnison Canyon involved some 30% grades and working conditions in the tunnel were difficult and dangerous because the tunnel was located at depths as great as 2,135 ft below the surface. As many as 800 people were employed on the tunnel in 1906, and twenty-six lives were lost due to a series of accidents during its construction. The tunnel was driven through granite, quartzite, gneiss, and shale as well as layers of sandstone, coal, and limestone. Excessive temperatures and humidity hampered the work. High levels of carbon dioxide, derived from inflows of warm groundwater, added to the dangers. In December 1906, warm water surcharged with carbonic acid was encountered in a fault zone, forcing drillers to abandon the heading for six months until a ventilation shaft was driven into the mountain. Almost one year was required to extend the tunnel through 2000 ft of water-filled rock. The water and humidity caused deterioration of tunnel lining timbers, making it necessary to line the tunnel with concrete.

The Gunnison Tunnel was completed at a cost of $2,905,307 and on September 23, 1909, President Taft was the guest of honor and primary speaker at dedication ceremonies held in the lavishly decorated town of Montrose. With a length of 30,650 ft, the Gunnison Tunnel was the longest irrigation tunnel in the world at that time (Clark and Simonds, 1994). The Uncompahgre Project was completed in 1925, at a total cost of approximately $6,800,000. In 1972, the American Society of Civil Engineers declared the Gunnison Tunnel a National Historic Civil Engineering Landmark. In 1979, the Gunnison Tunnel was listed on the National Register of Historic Places.

Colorado-Big Thompson Project: In 1904, the newly established United States Reclamation Service (USRS) concluded a report that suggested raising the elevation of Grand Lake 20 ft, creating a reservoir storing about 140,000 acre-ft of water. The plan included construction of a 12 mi tunnel from Grand Lake to either the Big Thompson River or St. Vrain Creek. When Congress created Rocky Mountain National Park in 1915, it specifically granted permission for the USRS to "enter upon and utilize for flowage or other purposes any area within said park which may be necessary for the development and maintenance of a Government Reclamation Project" (for additional information, see USBR WEBSITE references).

Subsequently built between 1938 and 1957 by the U. S. Bureau of Reclamation, the Colorado-Big Thompson (C-BT) Project is the largest transmountain water diversion project in Colorado. The C-BT Project annually delivers up to 310,000 acre-ft of water to northeastern Colorado for agricultural, municipal and industrial uses. It provides supplemental water to 30 cities and towns, and irrigates more than 600,000 acres of northeastern Colorado farmland. The entire project contains more than 100 major features, 125 water user organizations, 60 reservoirs and many distribution canals. The Northern Colorado Water Conservancy District (NCWCD) operates these facilities and maintains extensive information on their Website (for additional information, see NCWCD WEBSITE references).

The C-BT system has several important engineering geological features (Merriman, et al., 1957; Merriman, 1960). Twelve reservoirs, 35 mi of tunnels, 95 mi of canals and 700 mi of transmission lines are part of the complex collection, distribution and power system that spans 250 mi east to west and 65 mi from north to south.

West of the Continental Divide, Willow Creek and Shadow Mountain reservoirs, Grand Lake and Lake Granby collect and store the water of the upper Colorado River. The water is pumped into Shadow Mountain Reservoir where it flows by gravity into Grand Lake. From there, the 13.1 mi long Alva B. Adams Tunnel transports the water 3,800 ft beneath the Continental Divide to the East Slope.

Once the water reaches the East Slope, it is used to generate electricity as it falls almost half a mi through five power plants on its way to Colorado's Front Range. Carter Lake, Horsetooth Reservoir and Boulder Reservoir store the water. C-BT water is released as needed to supplement native water supplies in the South Platte River basin.

Fryingpan –Arkansas Project: Studies by the U. S. Bureau of Reclamation for a transmountain diversion project in south-central Colorado began in 1936. Intensive investigation started in 1941, resulting in potential planning reports in 1947 and 1948, followed by a special report in 1949 and official recommendations in 1951. A revised planning report under the name "Fryingpan-Arkansas Project" in 1953 led to congressional approval of the project. In September 1959, Congress authorized Ruedi Dam and Reservoir instead of the Aspen Dam and Reservoir.

The Fryingpan-Arkansas Project diverts on average 69,200 acre-ft of water annually from the Fryingpan River and other tributaries of the Roaring Fork River on the western slope to the Arkansas River basin on the eastern slope. This diverted western slope water, together with available water supplies in the Arkansas River Basin, provides an average annual water supply of 80,400 acre-ft for municipal/domestic use and for the supplemental irrigation of 280,600 acres in the Arkansas Valley. The project also includes one power plant with a generating capacity of 200 megawatts (for additional information, see USBR WEBSITE references).

There are two distinct areas of the project: the western slope, located within the Hunter Creek and Fryingpan River watersheds in the White River National Forests at elevations above 10,000 ft, and the eastern slope in the Arkansas Valley. Ruedi Dam and Reservoir, on the Fryingpan River, is the only project facility on the western slope. A great deal of geological and engineering debate occurred concerning the presence of evaporates subjacent to the Ruedi Dam site. Four dams and reservoirs are located on the eastern slope. Sugar Loaf Dam and Turquoise Lake, Mt. Elbert Forebay Dam and Reservoir, and Twin Lakes Dam and Reservoir are in the upper Arkansas watershed. Pueblo Dam and Reservoir, the largest reservoir in the project, is on the Arkansas River. The Charles H. Boustead Tunnel conveys all the water collected on the western slope under the Continental Divide to Turquoise Lake. The 10.5-ft-diameter, horseshoe-shaped tunnel is approximately 5.4 mi long.

Construction of the project began with Ruedi Dam and Reservoir in 1964. Project water for irrigation and municipal and industrial purposes was available in September, 1975 and the first electricity was produced at the Mount Elbert power plant in 1981.

Blue Mesa, Morrow Point, and Crystal Dams on the Gunnison River: Between 1962 and 1976, the U.S. Bureau of Reclamation constructed three dams and power plants along a 40 mi section of the Gunnison River (Blue Mesa, Morrow Point, and Crystal). Each of these dams has some noteworthy engineering geology features (for additional information, see USBR WEBSITE references, from which the following information was obtained).

The Bureau operates and manages these three dams as a unit. Initially named the Curecanti Unit, in November 1980 it was renamed the Wayne N. Aspinall Storage Unit in honor of former Congressman Aspinall, a strong advocate of water resource development in the west. The unit is part of the Colorado River Storage Project, which provides for the comprehensive development of the Upper Colorado River Basin. The project furnishes the long-time regulatory storage needed to permit States in the upper basin to meet their flow obligation at Lees Ferry, Arizona, as defined in the Colorado River Compact, and still utilize their apportioned water.

Constructed between 1962 and 1965, Blue Mesa Dam is on the Gunnison River about 30 mi below Gunnison, and 1.5 mi below Sapinero, Colorado. The zoned earth fill embankment has a structural height of 390 ft, a crest length of 785 ft, and a volume of 3,080,000 cubic yds of materials (Figure 12.).

Figure 12. Blue Mesa dam and power plant (U.S. Bureau Reclamation photo).

The spillway consists of a concrete intake structure, a concrete-lined tunnel, concrete flip bucket structure, and stilling basin. Maximum discharge of the spillway is 34,000 cubic ft per sec. Blue Mesa Reservoir has a total capacity of 940,700 acre-ft and an active capacity of 748,430 acre-ft. At maximum water surface elevation, the reservoir occupies 9,180 acres. It is one of the largest water bodies in Colorado and is an important recreational asset. The Blue Mesa power plant has a generating capacity of 60,000-kilowatts.

Constructed between 1963 and 1968, Morrow Point Dam, 12 mi downstream from Blue Mesa Dam, is the first thin-arch, double-curvature dam designed and constructed by the U. S. Bureau of Reclamation. The dam is 468 ft high, 52 ft thick at the base, and 12 ft thick at the crest, with a crest length of 720 ft, and a volume of 360,000 cubic yds. The spillway consists of four orifice-type openings in the top central part of the dam, providing a free-fall discharge of over 350 ft into the concrete stilling basin at the toe of the dam (Figure 13.). Maximum capacity of the spillway is 41,000 cubic ft per sec. Maximum reservoir capacity behind Morrow Point Dam is 117,190 acre-ft; the active capacity is 42,120 acre-ft. Surface area of Morrow Point Reservoir is only 817 acres, because it lies in the deeply entrenched and narrow Black Canyon of the Gunnison River (now a National Park).

Figure 13. Two spillways operating at Morrow Point Dam (U.S. Bureau Reclamation photo).

The Morrow Point power plant is unique within the Bureau of Reclamation. It is an underground chamber tunneled into the canyon wall in the left abutment about 400 ft below the surface. The chamber is 231 ft long and 57 ft wide with a height ranging from 65 to 134 ft. Additional engineering geological studies were required to evaluate a fault encountered at one end of the chamber. The power plant has a generating capacity of 120,000-kilowatts. (Figure 14.)

Constructed between 1973 and 1976, Crystal Dam is located 6 mi downstream from Morrow Point Dam and approximately 20 mi east of Montrose, Colorado. The dam is a double-curvature thin-arch type, 323 ft high, with a crest length of 620 ft, and a volume of 154,400 cubic yds. The reservoir has a total capacity of 25,236 acre-ft and an active capacity of 12,891 acre-ft, with a surface area of 301 acres. The power plant has a generating capacity of 28,000 kilowatts.

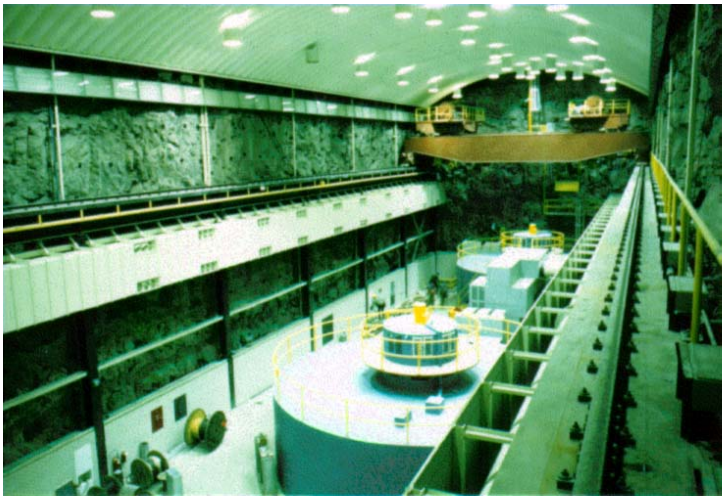

Figure 14. View of Morrow Point underground power plant (U.S. Bureau Reclamation photo).

Modernization of Horsetooth Reservoir: Horsetooth Reservoir, a component of the C-BT project located immediately west of Fort Collins, constructed in the 1950's, is unusual in that it is placed behind a hogback with four dams closing gaps (Figure 15). Because so many people live downstream from these dams, they must be brought up to standards of the 1984 Safety of Dams Act. All four dams are being modernized. In addition, treatment of seepage under Horsetooth Dam has been included in this work. The estimated cost for the entire project is $105 million and it will extend from March 2001until September 2003. Work is being conducted in three phases. Phase I, installation of filter and buttress at Horsetooth Dam, was completed during the summer of 2002. Phase II, installation of filters and buttresses at three eastern dams, is planned for completion by the fall of 2003. Phase III, addressing the seepage under Horsetooth Dam, was completed in November 2002 for additional information, see NCWCD WEBSITE references).

Figure 15. Horsetooth Reservoir under construction circa 1950 (U.S. Bureau Reclamation photo).

The most interesting engineering geology aspect of this work is the identification and remediation of cavities detected within the Forelle Limestone of the Lykins Formation underneath the reservoir that have produced seepage under Horsetooth Dam. Sonar tests done in 1999 indicated "anomalies" within the reservoir bed where limestone had collapsed or erosion had occurred. After the reservoir was drawn down to dead storage, one of these features was confirmed as a solution cavity (sinkhole). It was plugged with concrete and capped with plastic and clay before the reservoir began rising again. No other sinkholes were found near the dam.

The design for addressing seepage under Horsetooth Dam was changed to utilize an upstream blanket rather than the originally planned cut-off wall (Figure 16.). In addition to being a more effective "fix" for the seepage, installation of the upstream blanket saved time and was cheaper by approximately $19 million.

Figure 16. In October of 2002, an upstream blanket was placed along the reservoir bottom just south of Horsetooth Dam (U.S. Bureau Reclamation photo).

Ground Water Issues

Beyond the urban centers of the Front Range, ground water is the critical water resource (Colorado Geological Survey, 2002b). Ground water currently supplies about 18% of Colorado's needs. Nineteen of Colorado's 63 counties rely solely on ground water for potable supplies and domestic uses (Colorado Geological Survey, 2002b).

Reliance on ground water is increasing throughout Colorado. Recognizing the importance of this topic, the Colorado Geological Survey has issued, or assisted in the distribution of, several important statewide reports. The earliest report summarized the hydrogeologic conditions of all Colorado aquifers less than 2,000-ft deep (Repplier, et al., 1981). In 2001 the Colorado Ground-Water Association produced an atlas of ground-water resources in the State of Colorado, written for both laymen and ground-water professionals (Colorado Ground-Water Association, 2001). This publication provides some information on pertinent legislation and sources of information. In 2003 a new atlas was published (Topper, et al., 2003). This document comprehensively references Colorado's ground water resources, including specific information concerning the hydrological characteristics of the major aquifers and aquifer systems. Other major sources of Colorado ground-water information are:

- Colorado Division of Water Resources [http://water.state.co.us/],

- Ground Water Administration for the Office of the State Engineer [http://water.state.co.us/groundwater/groundwater.asp],
- Colorado District Office of the U.S. Geological Survey [http://co.water.usgs.gov/] and
- Colorado Department of Public Health and Environment's Water Quality Control Division [http://www.cdphe.state.co.us/wq/wqhom.asp].

Hydrogeologists and engineering geologists will no doubt continue to be employed in various aspects of water supply in Colorado, particularly in light of increasing environmental compliance issues, skyrocketing demand, and complex water management issues.

ENGINEERING GEOLOGY AND TRANSPORTATION

The earliest settlers, and the Native Americans before them, traveled on ft or by horseback. Even the early fur traders found the wide and shallow South Platte River a poor and difficult transport route. Early wagon trails tended to follow rivers and their valleys – more to ensure water supplies than for transport. As gold and silver exploration expanded into the mountains in the 1860's and 1870's, numerous rough wagon roads were constructed to connect the mining camps with smelters and supply centers. These were simply constructed without any engineering or geological evaluation and were often hazardous or impassable except in good weather (Figure 17).

Figure 17. Ouray-Silverton stage road, circa 1901 (SOURCE: Ben Mesander's Website: http://neurosis.hungry.com/~ben/colorado/rio-grande/)

Railroad Construction

The transcontinental railroad elected to by-pass Colorado for an easier crossing of the Rockies in Wyoming. In November 1867 the Union Pacific tracks reached Cheyenne, Wyoming. By 1868 the line had crossed Wyoming, and the transcontinental link was completed at Promontory Point, Utah, on May 10, 1869. Denver citizens were quick to search for ways to connect to this line. On May 18, 1868, the Denver and Pacific begin building north to Cheyenne from Denver. The first train arrived in Denver on June 23, 1870. Subsequently, the Kansas and Pacific arrived in Denver from the east, providing competition. These lines ran across relatively flat terrain on the eastern plains and paid little or no attention to geological conditions.

After 1870, narrow-gauge (3-ft spacing) railroads rapidly expanded from Denver to access mining camps throughout Colorado. The narrow gauge made construction easier and cheaper in mountainous terrain. There was fierce competition and most of the larger mining camps had more than one railroad providing access. Most narrow gauge railroad construction occurred between 1870 and 1895, with narrow gauge mileage in Colorado peaking at 11,699 mi in 1885 and declining thereafter (Hilton, 1990). This resulted from the passage of the Sherman Silver Purchase Act in 1890, leading to the "Silver Panic" in 1893. When the price of silver collapsed, many mining operations closed and the railroads lost their revenues.

The narrow gauge railroads avoided expensive solutions, such as tunnels, as much as possible, and "engineering geological" issues were rarely addressed. As a consequence the lines suffered from floods and landslides. Several notable engineering solutions were employed to resolve accessibility problems by these railroads. For example, the Colorado Central extended up Clear Creek to reach Georgetown in 1877, and the desire to overcome the rapid elevation difference to the lucrative Silver Plume mine and beyond led to the design and construction of the Georgetown Loop in 1884.

The Alpine Tunnel: At a few locations, the narrow gauge did undertake significant engineering geology projects. The Alpine Tunnel is probably the most famous; it allowed trains to pass under the Saguache Range, connecting Buena Vista to Gunnison, Colorado. At an altitude of 11,523 ft, and with a length of 1,772 ft, it became the first tunnel constructed across the Continental Divide. Tunnel excavation began in January 1880 and was expected to take only six months. However, due to unforeseen circumstances and with construction starting in the dead of winter, the task required nearly two years to complete. Fractured granite necessitated the expense of using over 400,000 board-ft of California redwood to support and line 1,427 ft of the 1,772-ft long tunnel at a total cost of around $300,000. Trains used the Alpine Tunnel for about 30 years – from July 1882 until November 1910 (Helmers, 1963). Today, the abandoned tunnel is the focus of the Alpine Tunnel Historic District (Figure 18), which has a website at http://www.narrowgauge.org/alpine-tunnel/html/.

Figure 18. The Alpine Tunnel Commemorative Marker near the West Portal (photo from Alpine Tunnel Historic District website).

The Moffat Tunnel: In 1902 David Moffat formed the Denver Northwestern and Pacific, soon reorganization as the Denver and Salt Lake (D&SL), to provide a direct westward standard gauge railroad route from Denver. The railroad crossed the Continental Divide at Rollins Pass, west of Boulder. The Rollins Pass route involved extremely steep grades (the maximum was 4.5%) and crested at 11,660 ft, and so was very difficult and expensive to operate. After much public debate, the State of Colorado created a special Moffat Tunnel Commission to fund a 6-mi long tunnel at an elevation of 9,249 ft that eliminated some 27 mi of average 3.5% grades. The new route had much lower grades and operating costs. In 1934, a short connection joined the D&SL with the Denver & Rio Grande Western main line at Dotsero, just east of Glenwood Canyon, thus providing a direct westward rail connection from Denver (Figure 19).

Figure 19. Train entering the western portal of the Moffat Tunnel

The Moffat Tunnel was built between 1923 and 1927, without benefit of adequate geological investigation. Based on a rushed surficial geological survey by the State Geologist, estimated costs were published and bids requested. Contractors were ultimately hired on a "cost-plus" basis, and even then the difficulties encountered delayed completion, resulting in financial penalties that eliminated all contractor profits. The ultimate cost was about four times the initial estimates (Lovering, 1928).

Preconstruction predictions were that the tunnel would be entirely in solid rock. In actual fact, about 2 mi on the west end were in weak ground that required substantial support. This section included about 1,000 ft in the Rock Creek Fault Zone that had extremely bad conditions; closure rates of up to 3 in per day required reinforcement of up to 2 tons of steel per running ft within concrete lining 30 to 45 in thick. A smaller pilot bore, located some 75 ft to the south, materially assisted the construction of the main tunnel by more accurately identifying rock conditions (Lovering, 1928). This pilot bore later became part of the first Denver water supply trans-mountain diversion project.

No geologists were involved during the Moffat Tunnel construction. The work was performed by civil engineers, without advice from mining engineers familiar with local mining operations. Subsequent official reviews concluded that the Rock Creek Fault Zone was not conspicuous on the surface and that potash-bearing montmorillonite clays were responsible for the instability of the worst ground in the fault zone (Lovering, 1928).

Thus the Moffat Tunnel was an early example of the value of having engineering geologists involved in major tunnels and similar projects. This lesson was followed in later construction of water supply and highway tunnels. Many of these projects experienced similar rock instability conditions. The water supply tunnels have been described previously. Highway tunnels are described in the following sections.

Early Highway Tunnels

Beginning in the 1940's a series of short highway tunnels were constructed in various locations within Colorado. Limited engineering geological advice was applied to these projects.

US 6 Clear Creek Canyon Tunnels: In the 1940's, following abandonment of the Colorado Central narrow gauge line west from Golden, U.S. Highway 6 was extended up Clear Creek Canyon. A series of six short tunnels were constructed to allow the road to pass through narrow sections of the canyon. The tunnels were mostly unlined and unlighted, however several have had additional concrete lining and illumination installed in recent years. These tunnels have performed well and have required only occasional maintenance.

Tunnel in Mesa Verde National Park: In the late 1950's it became necessary to relocate the only access road onto the Mesa Verde plateau using a 1476-ft long tunnel (Bohman, 1964). Instability of the existing road that angled upward across the high cliffs posed dangers to park visitors and required extremely expensive maintenance (Figure 20). Bohman reported that the tunnel was excavated in colluvium, weathered Mancos Shale, and dense, bluish to grey, unweathered shale. Considerable difficulties were encountered with raveling failures during the tunnel advance, although pre-construction investigations had suggested that the unweathered shale would not require temporary support. Work adjustments allowed the tunnel to be completed in 1957 without a major incident, including installation of a permanent concrete lining.

Figure 20. As this 1925 photo shows, the famous Knife Edge Road held certain terrors for early park visitors. Though eventually improved and surfaced, it was abandoned in 1957 when the new tunnel was completed (U.S. National Park Service photo).

Idaho Springs Tunnels: Interstate-70 west from Denver avoided the lower section of Clear Creek Canyon, but joined Clear Creek near Idaho Springs where a pair of short, fully-lined, and lighted tunnels were constructed in the late 1960's (Figure 21).

Figure 21. Eastbound view of the "twin tunnels" on Interstate 70 east of Idaho Springs (Photo by M. E. Salek http://www.mesalek.com/colo/index.html).

Interstate-70 Construction Activities

As had occurred with the transcontinental railroads almost a century before, the initial plans for the Interstate highway system in 1956 omitted any Interstate route across the Colorado Rockies. Active lobbying soon had Interstate 70 extending westward across Colorado to Grand Junction, then westward across eastern Utah to join with Interstate 15 in southern Utah. However several sections of this route were not explicitly defined, and the construction of an Interstate west from Denver across the Rockies raised several engineering geological and environmental challenges.

The route through the western Denver suburbs and into the foothills provided some initial controversy. For example, the "hogback cut" through the Dakota Hogback south of Golden was only approved after geologists proposed making it a "Point of Geological Interest" with parking facilities, walkways along the cut, and interpretative signs.

Progress was achieved so that by the early 1970's the Interstate extended west from Denver to the eastern approach to Loveland Pass across the Continental Divide. To the west, sections of Interstate were completed around Dillon, westward from Vail to Eagle, westward from Glenwood Springs, and in the Grand Junction area west to the Utah line. There remained four major gaps where engineering challenges or controversies concerning the appropriate route were causing delays. Closing the first of these gaps required solving the major challenge of designing and constructing the proposed tunnel under the Continental Divide to bypass Loveland Pass.

The second gap resulted from the controversy that surrounded the selection of the route between Dillon (Frisco) and Vail. The Colorado Department of Highways favored a route over Red Buffalo Pass, which was nearly 11 mi shorter than the existing highway over Vail Pass and avoided the difficult geological conditions of Vail Pass. However, Red Buffalo Pass lay within the Gore Range-Eagle Nest Primitive Area of the Arapaho and White River National Forests. This wilderness had been established in 1933, and already had been reduced in 1941 to accommodate U.S. Highway 6 over Vail Pass. In 1968, Orville Freeman, Secretary of Agriculture, denied the request for access to Red Buffalo Pass, basing his decision largely on supporting the integrity of wilderness areas. Subsequently, the Interstate route over Vail Pass became an example of the innovative designs and construction techniques used to preserve the sensitive and scenic mountain environment.

The third gap between Eagle and Glenwood Springs existed for two reasons. The section between Eagle and Dotsero at the eastern end of Glenwood Canyon involved construction in an area of evaporite deposits and adjacent to unstable slopes subject to debris flow hazards. This section thus required additional careful investigation and design, but was successfully completed in 1982. The Glenwood Canyon route became the most environmentally contentious section of Interstate 70. Several alternatives were considered, but eventually the decision was made to construct the route through the canyon. The Colorado Legislature called for "the wonders of human engineering to be blended with the wonders of nature". Construction of the Glenwood Canyon section was one of the most challenging projects in the entire Interstate Highway System – it produced a world-class scenic byway and an outstanding environmental and engineering project, but it was the final section of Interstate 70 to be completed (in 1993).

The fourth gap that remained in the mid-1970's was through another Colorado River Canyon, DeBeque Canyon, located east of Grand Junction. This canyon did not present as difficult engineering and environmental challenges as Glenwood Canyon, and the gap was closed by 1989. However, slope instability within DeBeque Canyon includes the extremely large DeBeque Landslide complex that reactivated in 1998 and threatened the Interstate. The engineering geological investigations of this landslide have encouraged Federal-State-Academic-Private

Sector collaboration, thereby further encouraging the development of engineering geology partnerships in Colorado.

The construction of Interstate 70 across the mountains west of Denver thus created many engineering geology opportunities. The sequence of three projects – Eisenhower Tunnel, Vail Pass, and Glenwood Canyon – were especially important to the development of the practice of engineering geology in Colorado. They spanned a period of almost three decades – from the mid 1960's to the early 1990's. During this period new exploration and construction technologies became available, and were experimented with, tested, and used on these projects to the point where they became adopted as standard practice. Environmental impacts became an important design constraint in this period also, and these projects reflect this change in societal demands.

This report can only summarize the most important engineering geology lessons learned from the Eisenhower Tunnel, Vail Pass, and Glenwood Canyon Projects. These projects were undertaken by a team of geologists and engineers employed by the Colorado Department of Highways, later the Colorado Department of Transportation (CDOT), with assistance from private consultant engineers and geologists, and by engineering geologists employed by the Colorado Geological Survey. Several members of this team began their careers on the Eisenhower Tunnel, then continued with the Vail Pass and Glenwood Canyon projects. It was remarkably fortunate that this group of talented individuals was able to apply the lessons learned on earlier projects to the increasingly sophisticated and complex design, construction, and management requirements posed by the later projects.

Eisenhower Tunnel: Officially named the Eisenhower-Johnson Memorial Tunnels, these "twin bore" tunnels each carry two lanes of traffic in one direction under the Continental Divide. Each bore is about 8,940 ft (1.69 mi) long. These tunnels are the highest part of the U.S. Interstate Highway system, and are probably the highest vehicular tunnel in the world. The official elevations are 11,013 ft at the east portal, 11,112 ft at the midway point, and 11,158 ft at the west portal. Because of this altitude, ventilation of vehicle exhausts is a major concern. Each bore has a very large egg-shaped cross-section with a height of 48 ft and a width of 40 ft. Huge exhaust and fresh air ducts bring the roadway's height down to 16 ft-4in, so that the typical tunnel user is unaware of their true size. Ventilation is provided by multiple large fresh air and exhaust fans that each can move over 500,000 cubic ft per minute. Large control and ventilation structures can be seen at each portal (Figure 22).

The large required tunnel cross-sections, high altitude, considerable length, and difficult rock conditions created engineering geological and construction challenges. Construction on the first bore, the westbound or north tunnel, took five years, between March 15, 1968 and March 8, 1973. This tunnel carried one lane of traffic in each direction until the second bore, the eastbound or south tunnel, was completed. This second bore was constructed in four years, between August 18, 1975 and December 21, 1979. The westbound bore was originally called the Straight Creek Tunnel, and subsequently was officially named the Eisenhower Memorial Bore, while the eastbound bore was named after Edwin C. Johnson, a past Colorado Governor and U.S. Senator who had actively supported an interstate highway system across Colorado.

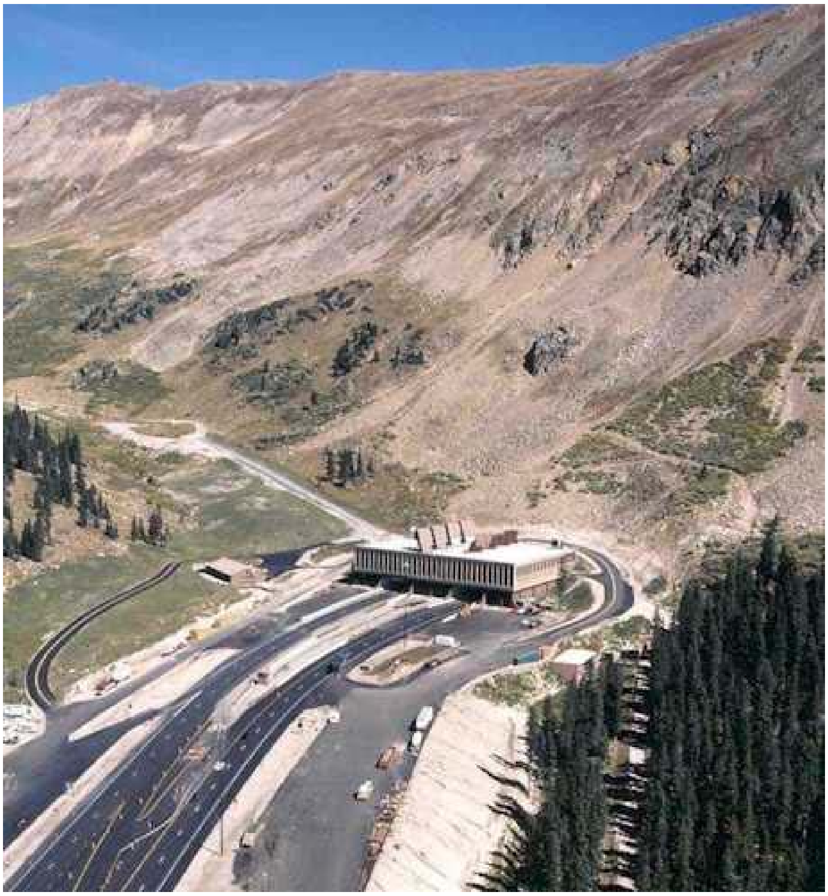
Figure 22. Aerial view of the western portal of the Eisenhower-Johnson Tunnels (Photo by Gregg Gargan, CDOT).

The Western approach to the tunnel is along Straight Creek, so named because it follows a fault zone. Slope instabilities along this section, combined with a very steep grade, actually exceeding normal Interstate standards, demanded careful geological studies prior to the design and construction of a six-lane roadway, allowing for slow speed truck lanes in both directions, and two truck escape ramps. The eastern approach has some 6% grades but has not required truck escape ramps. It has been subject to debris flows and some avalanche hazards. Engineering geological studies relating to the debris flows are described subsequently.

Vail Pass: The decision to abandon the Red Buffalo Pass option and construct Interstate 70 over Vail Pass in close proximity to the existing U.S Highway 6 alignment presented many challenges. The project became a landmark for engineering geology in Colorado because it was the first major project to utilize engineering geology advice throughout – from design to construction (Hynes, 1983a). Its technical success and pleasing aesthetics set the stage for many other projects with engineering geologists being key partners to the design and construction of difficult and challenging engineering works (Figure 23).

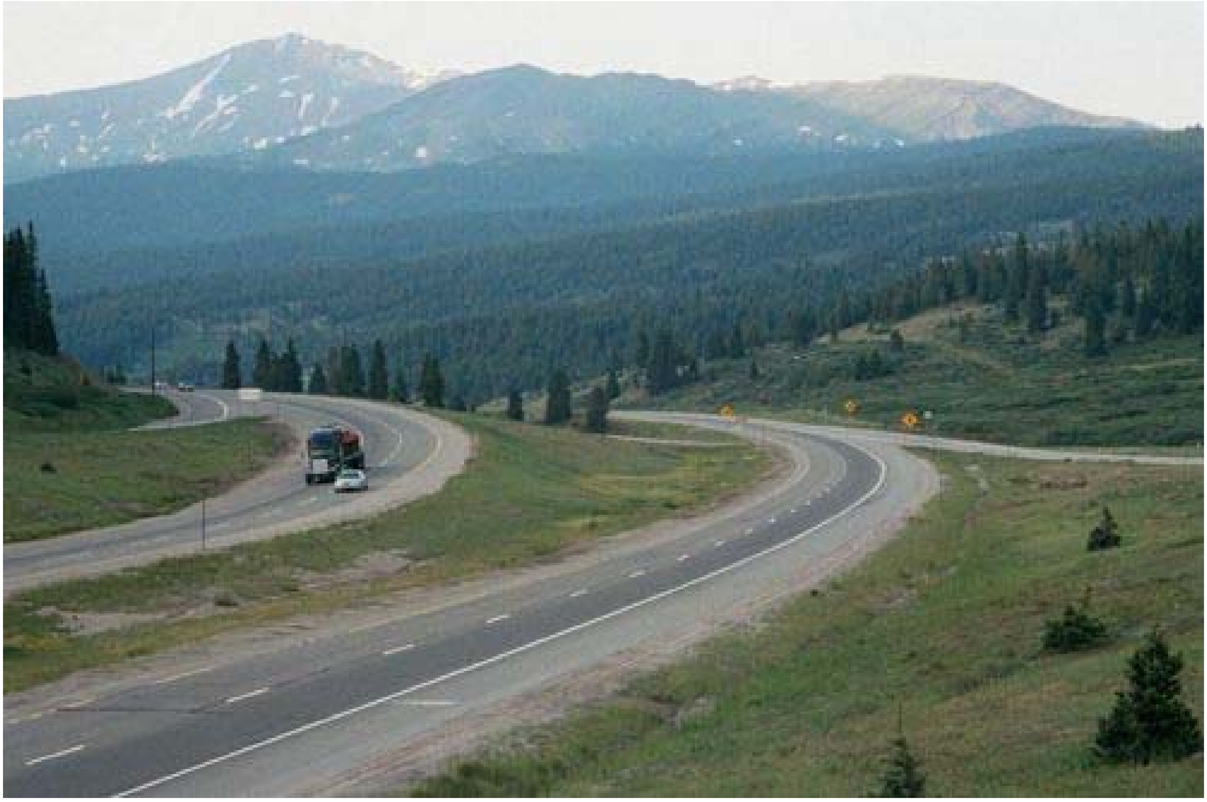

Figure 23. The Summit of Vail Pass showing the environmental integration of the highway with the surrounding landscape (Photo by M. E. Salek http://www.mesalek.com/colo/index.html).

Engineering geological studies were undertaken in 1970-1971, with additional design studies undertaken in 1972-1973 (Robinson and Cochran, 1983). Actual construction took several years and was completed in 1978. In addition to the difficult geological conditions, the project engineers faced requirements for keeping traffic moving over Vail Pass during construction, and stringent environmental requirements concerning erosion control and water quality.

The geological setting of Vail pass includes Precambrian igneous and metasedimentary rocks exposed east of the Gore Fault Zone and red arkosic sandstones, siltstones, and claystones of the Pennsylvanian-Permian Minturn and Maroon formations to the west of the fault zone. The preferred route over Vail Pass crosses the fault zone on the eastern side of the Pass, and parallels the fault zone on the western side of the Pass. Bedrock is sheared and structurally deformed over a considerable area, and glacial action has over-steepened many slopes on both sides of the Pass. Colluvium and glacial morain are found at close to their natural angles of repose on these slopes (Robinson and Cochran, 1983).

Consequently, landslides are common along the route, especially on the western side of Vail Pass where several sections of almost continuous landslides are found along Black Gore Creek. Additional problems resulted from the easily eroded character of the colluvial and glacial deposits, and the difficulty of stabilizing them once disturbed during construction, since re-establishing vegetation was problematic at these altitudes with its short growing season.

Many innovative concepts and designs were utilized or developed on this project. Slope stability and erosion control were paramount concerns. For much of the route, the East- and West-bound lanes were widely separated to reduce the sizes of required cuts and fills. Large rock cuts were avoided. Several comparatively long viaduct bridges were used to further reduce the size of fills. Because bridge construction had to minimize environmental disturbance, segmental pre-cast post-tensioned bridges were used at several locations for the first time in Colorado. Several innovative retaining wall designs were tested, and the first reinforced earth walls in Colorado were installed. Landslide controls included efforts to minimize the loadings imposed by the roadways, as noted above, and also a variety of drainage and slope sculpturing schemes. At one location on the western side, two landslides on opposite sides of the valley were stabilized by allowing them to buttress each other. Fill was added to the valley, the stream profile was raised and controlled to prevent further erosion of their toes, and the highway was constructed on the fill (Robinson and Cochran, 1983).

Engineering geologists coordinated with landscaping experts to produce a finished highway design that appeared to belong within the landscape and enhanced the visual experiences of roadway users. Measures partly dictated by stability concerns, including widely separated roadways and avoidance of large rock-cuts and fills, were further enhanced by sculpting rock cuts to appear more "natural-looking" and the use of colored concrete in bridge structures and retaining walls to blend with rock and soil tones. Careful revegetation, including the "planting" of old tree stumps on cut slopes located within an old forest-fire zone, and extensive slope contouring further enhanced the visual integration.

Lessons learned on Vail Pass were later applied to other Colorado highway projects. Between 1976 and 1978, re-alignment of Colorado Highway 91 north of Fremont Pass encountered similar geological conditions at high elevations and similar engineering geological approaches were used to produce a stable and economical solution (Ivey and Hanson, 1983; Holmquist, 1983). The Colorado Department of Transportation recently completed the Berthoud Pass Mountain Access Project that widened 6 mi of U.S. 40 on the east side of Berthoud Pass to three lanes: two lanes uphill, one lane downhill, between Berthoud Falls and the summit. This project encountered numerous slope stability and rockfall mitigation conditions that required environmental controls (Colorado Department of Transportation, 2003).

Glenwood Canyon: In 1982, after many years of pre-planning and environmental impact studies, construction began on the Glenwood Canyon I-70 segment (Bowen, 1988; Hynes, 1983b). Alternate routes over the Flat Tops Wilderness to the north and Cottonwood Pass to the south were evaluated, but were rejected as being even more difficult and involving even greater environmental impacts than the route through Glenwood Canyon.

Geologic hazards through Glenwood Canyon included severe rock fall problems, debris flows, landslides, unstable talus slopes, and other conditions discovered as the project progressed (Bowen, 1988; Hynes, 1983b). In addition to the inherent geological challenges, severe environmental and aesthetic constraints were placed on the project, including: minimal disturbance to pre-existing slopes, sculpting of rock cuts, and staining of fresh rock-cut scars to mimic natural weathered surfaces (Turner, 1986). Design of the roadway elements optimized aesthetics and visual impacts so as to maintain or improve traveler's impressions of this world-class canyon (Figure 24).

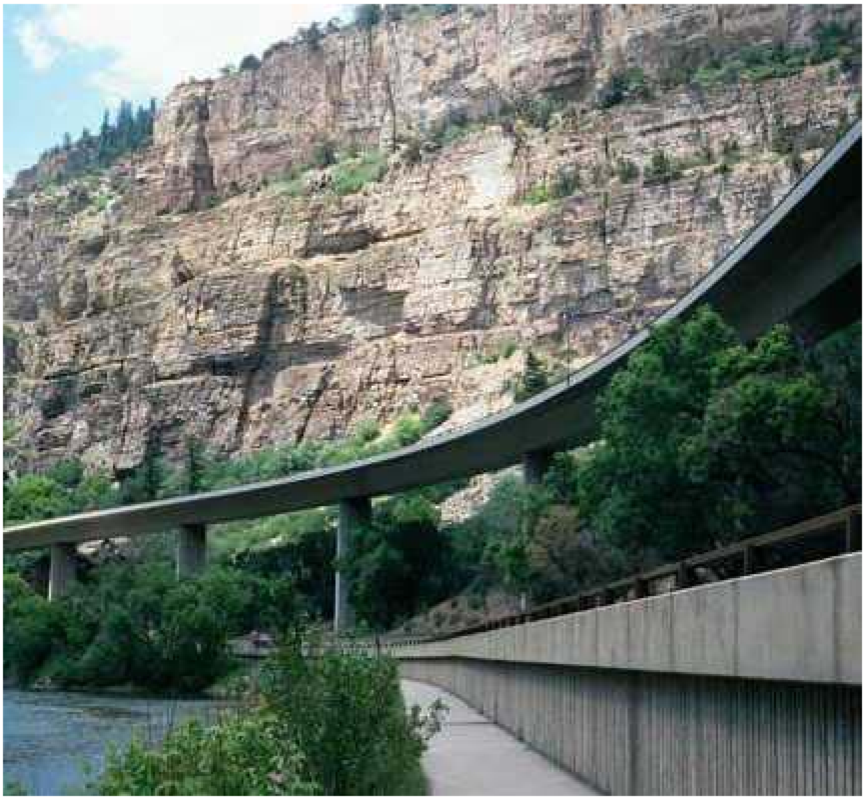

Figure 24. View of Interstate 70 in Glenwood Canyon.

In many cases, the engineering design solutions were extensions or expansions of experiences gained earlier on Vail Pass. Extensive lane separation allowed for reduced impact on the existing canyon environment, while also producing better aesthetics for the highway users. This resulted in the construction of 39 bridges ranging in length from 100 ft to 7000 ft (Figure 24). An architectural oversight committee had to review and approve all designs. Once again colored concrete elements were specified to blend with the natural tones of the canyon. Rock cuts could have no drill-hole half-casts, had to reflect natural benches, and had staining applied so that they matched weathered exposures.

Many bridges and other structures required footings in talus, and new exploratory methods in talus had to be devised, along with appropriate methods of compaction grouting of talus to prepare the foundations (Turner, 1996). The bridge decks were built with both segmental and cast-in-place cantilevered procedures so that disturbance to existing canyon ecosystems was minimized. Experiments were undertaken to prove the suitability of a variety of earth reinforced and tied-back retaining wall structures for both temporary and permanent installations (Bell, et al., 1983). Innovative designs included sophisticated cantilevered post-tensioned roadway slabs over retaining walls (Turner, 1986). Tunnels were used at critical sections, largely for environmental protection and aesthetics.

Geological exploration discovered a distinctive buried "gray-layer" in the alluvium of the eastern half of Glenwood Canyon. Geological interpretations suggested that this layer was deposited in a lake formed behind a rockfall dam located in the Canyon near the Shoshone power plant. This clay could potentially cause large foundation settlements, and design adjustments were required for several highway structures in the eastern part of the project.

The Colorado Department of Transportation began utilizing the Colorado Geological Survey in 1984 to provide engineering geological advice to the Glenwood Canyon project. Two Colorado Geological Survey engineering geologists supervised a staff of between 10 and 20 geologists, engineers and drilling personnel. Among their tasks was an assessment of rockfall hazards caused by the steep terrain and complex geology of the canyon environment. Colorado Department of Transportation personnel and Colorado School of Mines researchers developed a computer modeling approach called Colorado Rockfall Simulation Program (CRSP) that was used to evaluate innovative rock fall barrier designs. CRSP was calibrated with field experiments and subsequently updated and widely publicized (Pfeiffer, 1989; Pfeiffer and Higgins, 1990; Jones, et al., 2000; Higgins, et al., 2003).

ENGINEERING GEOLOGY AND GEOHAZARDS

The topography and geology of Colorado conspire to produce a wide variety of naturally occurring surface processes that become geohazards when human populations and infrastructure elements are placed in close proximity – landslides, rockfalls, debris flows, floods, problematic soils that may shrink, swell, or hydrocompact, and even moderate-sized earthquakes. During the 1960's and 1970's, geohazards increased as a concern to all levels of society as populations expanded into high-risk areas, prompting construction of elaborate water supply systems and new transportation facilities, such as Interstate 70, across the mountains. At the same time, these actions were subjected to increased scrutiny on diverse environmental issues. The following sections describe critical milestones in the application of engineering geology to these geohazards. Slope instability processes are covered under the topics of landslides, rockfalls, and debris flows. Engineering geology assessments of geohazards associated with floods, problematic soils, subsidence caused by abandoned mine, and our limited experiences with earthquakes, round out this section. Human, legislative, and organizational aspects of engineering geology, which are closely related to these geohazards and responses to them, are discussed in the next major section of this paper.

Landslides

The earliest recorded scientific description of a Colorado landslide appears to be the description of the Slumgullion landslide by Endlich in the report of the Hayden Survey for 1874 (Endlich, 1876). The earliest reported landslide event that resulted in directed field investigation and a scientific report was the Cimarron Landslide of 1886 (Cross, 1886).

Beginning in the 1970's, landslide studies became more important due to population expansion in high-risk areas and construction of Interstate 70 across the mountains, coupled with an increased focus on diverse environmental issues. Engineering geologists were called upon to evaluate numerous landslides. Many of these projects were sponsored by the Colorado Geological Survey or by the U. S. Geological Survey. In 1988, Colorado developed a statewide landslide hazard mitigation plan (Jochim, et al., 1988). This plan contained a list of 49 "vulnerable communities, areas, and facilities", and this list has recently been revised and updated (Rogers, 2003).

The scope of the landslide phenomena and their distribution in Colorado is so great that an inclusive discussion of the history of landslides investigations in Colorado is impossible. The following sections merely highlight some interesting and noteworthy landslide topics.

Early Landslide Investigations in the San Juan Mountains: In 1909, the U. S, Geological Survey published a professional paper on the landslides in the San Juan Mountains of southwestern Colorado (Howe, 1909) that demonstrates an interest in engineering geology subjects during the initial geological investigations of Colorado. Howe's report is based on some 10 years of landslide studies, undertaken as part of more extensive geological investigations of the San Juans, an important silver and gold producing area with several mining centers. By 1909, geological studies in this region had been undertaken for over 30 years, and the U.S. Geological Survey had published six of their in-depth Folios. Howe reports on earlier landslide investigations, including the landslide that occurred on Cimarron Creek in 1886. This Cimarron Landslide was discovered by a rancher within days of its occurrence and is probably the first landslide in Colorado to be subjected to field investigation and reporting (Cross, 1886).

More importantly, Howe's report can be considered the earliest engineering geology report dealing with a Colorado subject. Howe supplements his descriptions of landslides with a summary of the current understandings of landslide mechanisms and processes, referencing landslide investigations in Europe and the Frank, Alberta landslide in Canada. In this way, Howe provided insights concerning early engineering geology concepts – he proposed a classification for landslides in the San Juan Mountains after evaluating a landslide classification by Heim based on observations in the Alps.

Studies of the Slumgullion Landslide: The Slumgullion landslide (Figures 25 and 26) in the San Juan Mountains near Lake City, Colorado is perhaps the most famous Colorado landslide. It is also one of the largest, being about 4.5 mis long and having an estimated volume of 220 million cubic yards (Parise and Guzzi, 1992). Numerous investigators have studied it (Endlich, 1876; Howe, 1909; Atwood and Mather, 1932; Burbank, 1947; Crandell and Varnes, 1960, 1961; Keefer and Johnson, 1983; Fleming, et al., 1996; Varnes and Savage, 1996; Parise and Moscariello, 1997).

The Slumgullion landslide is a combination earth flow and earth slide that is composed of altered Tertiary volcanic rocks. It consists of two parts: a younger, active, upper part that is over-riding a lower, older, much larger, inactive part. Based on radiocarbon and tree-ring dating, the younger, active part is about 300 years old, while the older, inactive part, which dammed the Lake Fork of the Gunnison River creating Lake San Cristobal, is about 700 years old (Crandell and Varnes, 1960; 1961).

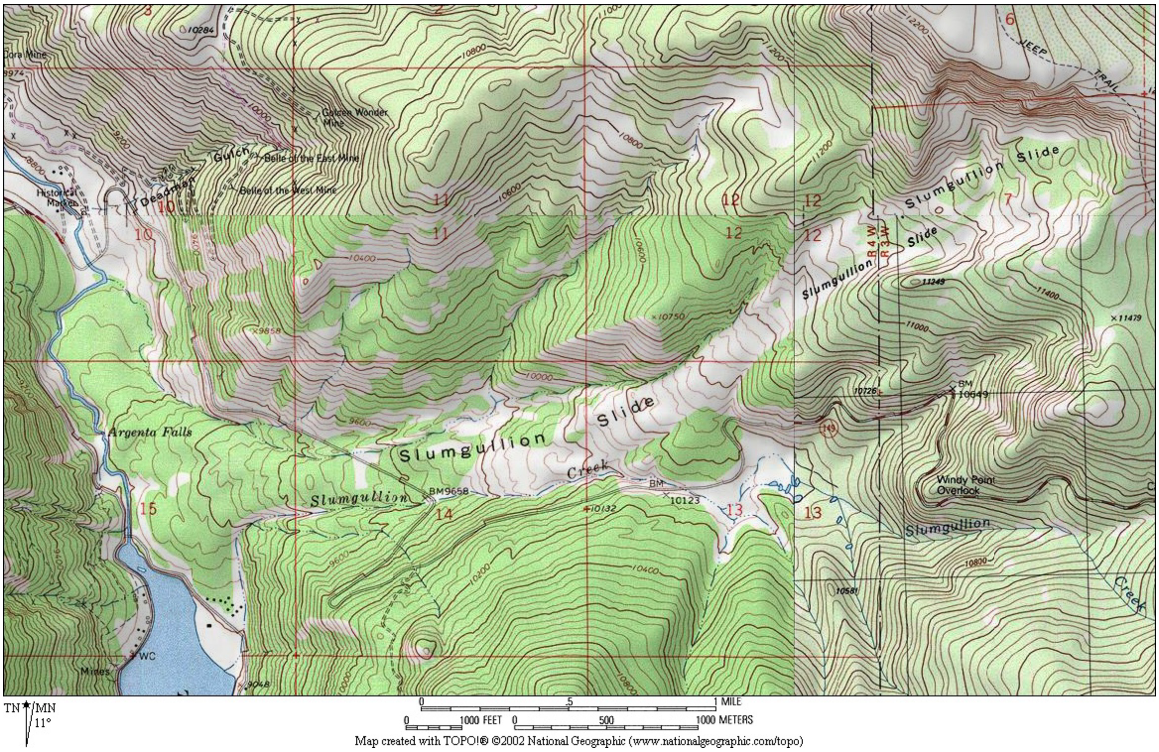

Figure 25. Map of the Slumgullion landslide.

Figure 26. Slumgullion landslide with Lake San Cristobal at lower right (U.S. Geological Survey photo).

State Highway 149 connecting Lake City and Creede crosses the lower part of the landslide, which is increasingly occupied by condominiums and other structures near Lake San Cristobal. This part of the slide mass is believed to be stable, based on very limited evidence such as the vertical attitudes of trees and absence of major distress on the highway. However, some evidence suggests that the active upper part may be surcharging the lower part, so that long-term stability cannot be assured (Fleming, et al., 1996; Varnes and Savage, 1996; Parise and Moscariello, 1997).

In 1990, the U.S. Geological Survey, in cooperation with scientists sponsored by the Italian National Research Council, began a new study of the Slumgullion landslide (Varnes and Savage, 1996). It was chosen for a comprehensive study of landslide processes because movement data have been collected periodically for more than 30 years. Slumgullion kinematic, geophysical, geotechnical, and hydrological data will be combined into quantitative models for hillslope stability that may be applied to other locations (Varnes and Savage, 1996).

Dowds Junction Landslides: Four large old landslide masses are located near the junctions of U.S. Highways 6 and 24 with Interstate 70 a short distance west of Vail (Figure 27).

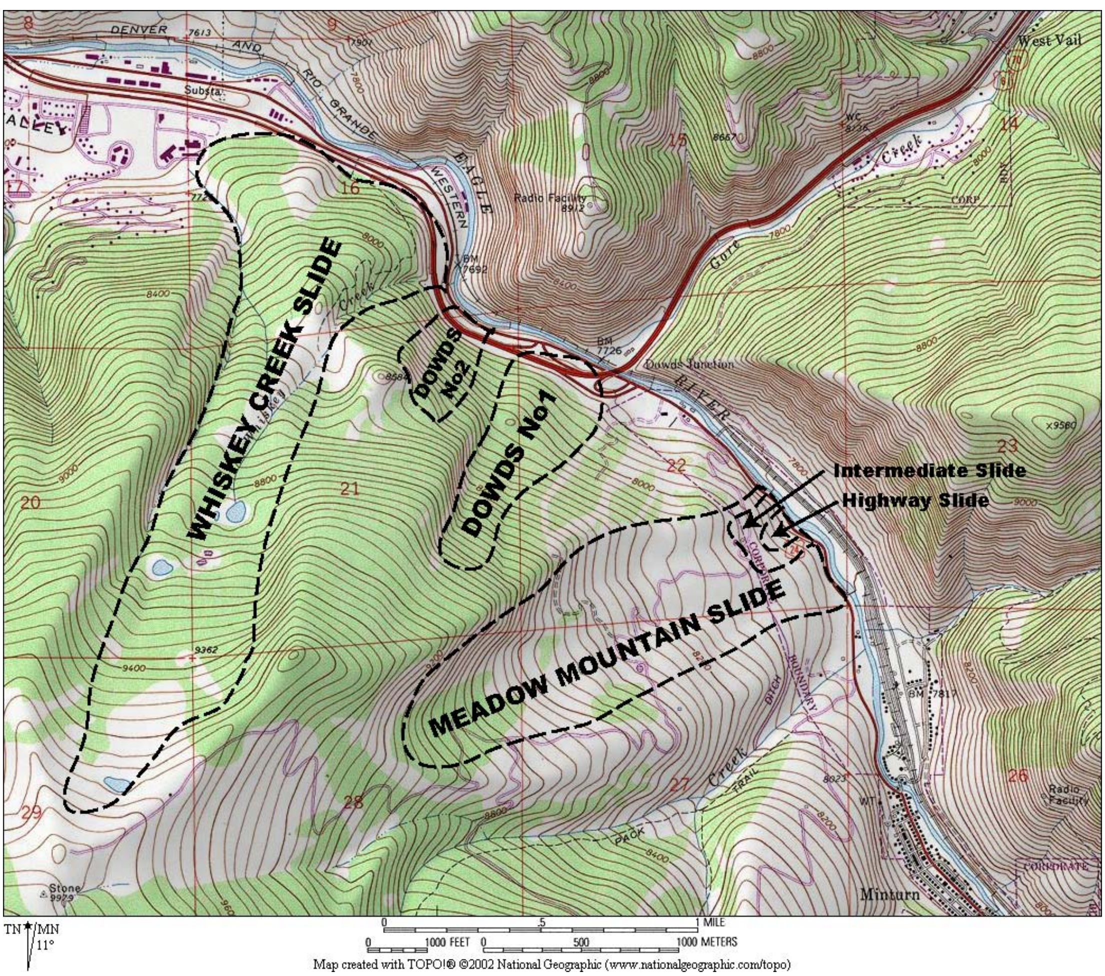

Figure 27. The Dowds Junction Landslide Complex (modified after Soule, 1988 Fig.1, and Jochim et al., 1988, Fig. 19).

A lengthy history of highway maintenance problems has resulted from reactivation of these landslides by highway construction – the Meadow Mountain Landslide has necessitated continuing maintenance to U.S. Highway 24 since its construction in 1930. The design of Interstate 70 through this area recognized these landslides, but continuing maintenance has been required. The State of Colorado declared a Landslide Alert at this location in 1985 and it subsequently was used as one of three case studies in the state landslide hazard mitigation plan (Jochim, et al., 1988). The Landslide Alert resulted in extensive geologic investigations, monitoring, and emergency exercises at Dowds Junction.

The chief concern was that a major landslide could dam the Colorado River, causing inundations upstream at Vail and Minturn, and subsequent downstream flooding when the dam was overtopped or failed. In addition, major failures could disrupt the Interstate 70 and/or the Highway 24 routes and the Denver and Rio Grande Western rail corridor. As a consequence, considerable effort was made to evaluate these landslides, including installation of a variety of monitoring and instrumentation systems (Soule, 1988). During the monitoring period, no catastrophic failures occurred and some parts of the monitoring systems were allowed to lapse, but EDM stations continue to be measured annually. Duran undertook additional investigations subsequently (Duran, 1993).

East Muddy Creek Landslide: In the spring of 1986, a part of an extremely large Quaternary landslide complex reactivated in response to exceptionally high precipitation in the preceding three years and the wettest winter and spring on record for this area (Stover, 1986; Stover and Cannon, 1988). The central of three landslides reactivated (Figure 28) and began moving at historically unprecedented rates of up to 10 in per hour. The landslide effectively destroyed about a mi of Colorado State Highway 133 and constricted the channel of East Muddy Creek. Since it was located only a short distance upstream of Paonia Reservoir (Figure 28), initial fears were that a large landslide dam might form which would have threatened the safety of the reservoir and its dam (Stover and Cannon, 1988).

Fortunately, emergency crews working around the clock for five weeks were able to raise and realign the road and maintain the flow volumes of East Muddy Creek, preventing more serious consequences. Once again experience was gained with field installation and operation of a variety of movement monitoring equipment (Stover and Cannon, 1987a; 1987b). This landslide event was judged unique enough, primarily due to its size, rate of movement, and successful mitigation, to be presented as a case study to an international audience (Stover and Cannon, 1999). Subsequent studies evaluated the impact of this landslide on sediment production and sediment loadings in Paonia Reservoir (Appel and Butler, 1991).

Landslide Hazards along the Front Range: Landslides are fairly common at several Colorado Front Range locations. Given the population concentration and the many transportation and utility corridors that are threatened, these landslide hazards have received considerable attention from engineering geologists for many years. Landslides have been concentrated on the slopes of North and South Table Mountains in Golden, on Green Mountain to the south of Golden (Figure 29), and several older landslides have caused damage in and near Colorado Springs. FEMA sponsored a study of landslide hazards in Colorado Springs that resulted in FEMA purchasing many properties that were damaged by landslides (Wait and White, 2002).

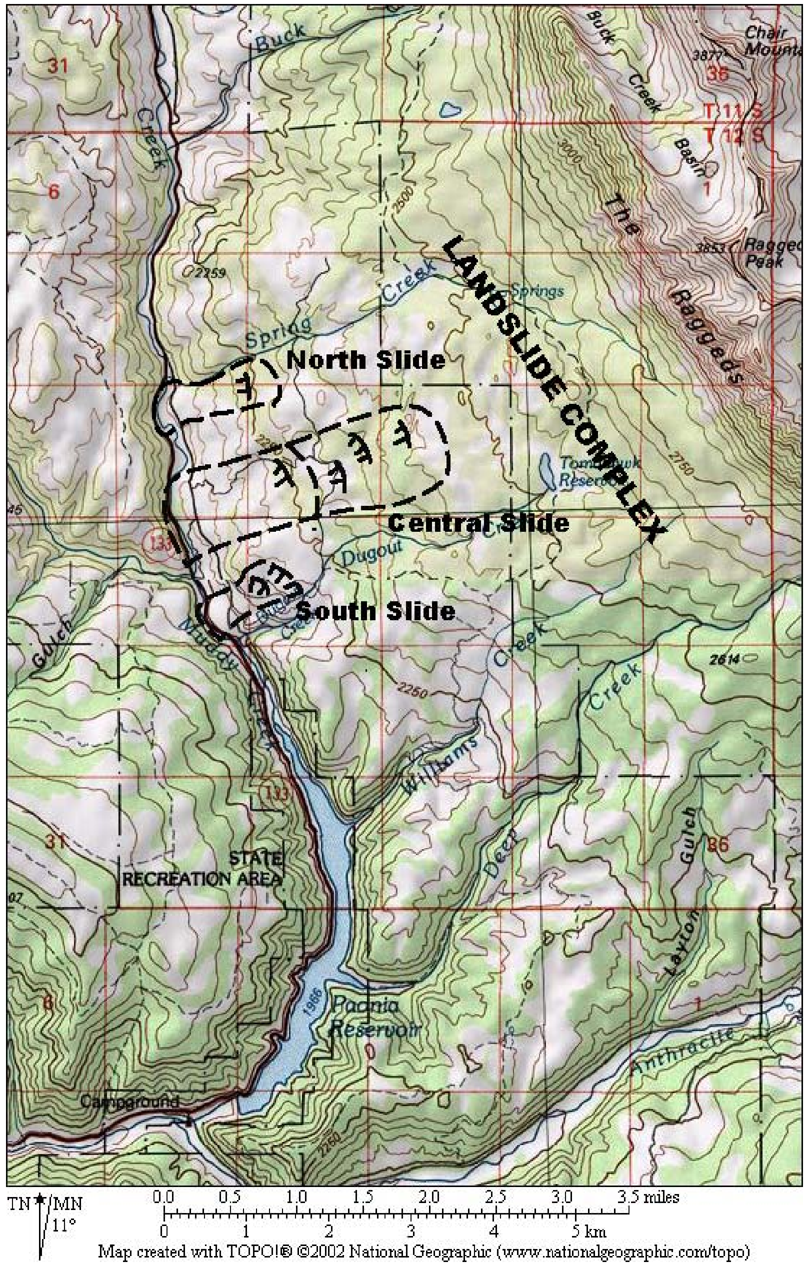

Figure 28. Map of the East Muddy Creek Landslides (modified after Stover and Cannon, 1988, Figure 1b, and map by Stover, 1986).

Engineering geology maps, and in some cases landslide maps, were prepared for the Golden and Morrison areas (Gardner, et al., 1971; Scott, 1972; Van Horn, 1972; Simpson, 1973; Simpson and Hart, 1980). In addition, in 1975 the U. S. Geological Survey published three regional landslide maps covering the entire Front Range region (Colton, et al., 1975a, 1975b, 1975c). Other similar maps were produced for other locations, most recently for Colorado Springs (Scott and Wobus, 1973; Carroll and Crawford, 2000; Wait and White, 2002).

In spite of these efforts, landslides have continued to cause considerable damage to residential developments along the Front Range Urban Corridor. Over 10,000 new homes are built each year, and some are located in landslide hazard zones. For example, in the City of Colorado Springs, 300 houses are located on a recently active landslide and about 5000 homes are located on older landslides (Wait and White, 2002). In 1995, a landslide affected two Colorado Springs homes with the result that both houses were condemned and their owners sued the developer. In 1998, after a wet spring, a landslide affected five houses on the northwest portion of Green Mountain, and two houses were ultimately condemned. These houses were constructed on identified and mapped landslides.

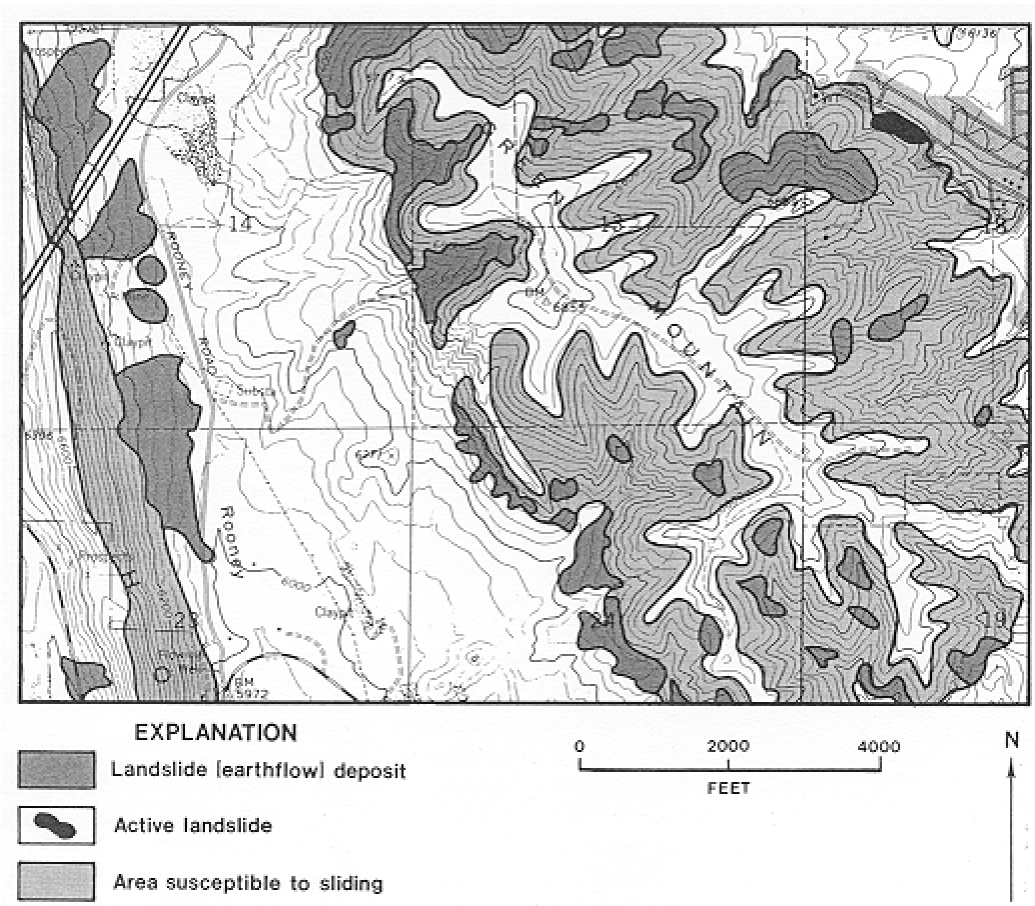

Figure 29. Map of areas susceptible to landsliding in the Green Mountain area of the Morrison Quadrangle (Detail of map by Scott, 1972).

DeBeque Canyon Landslide: The DeBeque Canyon Landslide is a major landslide complex that has historically impacted the east-west highway and railway corridor on the Colorado River in western Colorado (Stover, 1988b). The earliest known major ground movement occurred in the late 1890s or early 1900s and reportedly shifted the river channel and damaged railroad facilities on the north bank of the Colorado River (Miller and Higgins, 2000). In February 1958, after widening of the 2-lane roadway undercut and destabilized the landslide toe, movements heaved the roadway upward 23 ft. In April 1998, the renewed movements caused Interstate 70, constructed in the mid-1980s, to heave 14 ft and shift laterally 5 to 6 ft towards the river (White, 2003).

In response to the 1998 event, and with concern about possible future catastrophic events, the Federal Highway Administration funded a multi-agency team of geologists and engineers to investigate and monitor the landslide, and to propose mitigation concepts for Interstate 70. The team, administered and managed by the Colorado Department of Transportation Geotechnical Unit, included personnel from the Colorado Geological Survey, Golder Associates, Inc., the Colorado School of Mines, and the United States Geological Survey (Colorado Geological Survey, 2000; White, 2003). The team investigators divided the DeBeque Canyon Landslide into several components or zones (Figure 30).

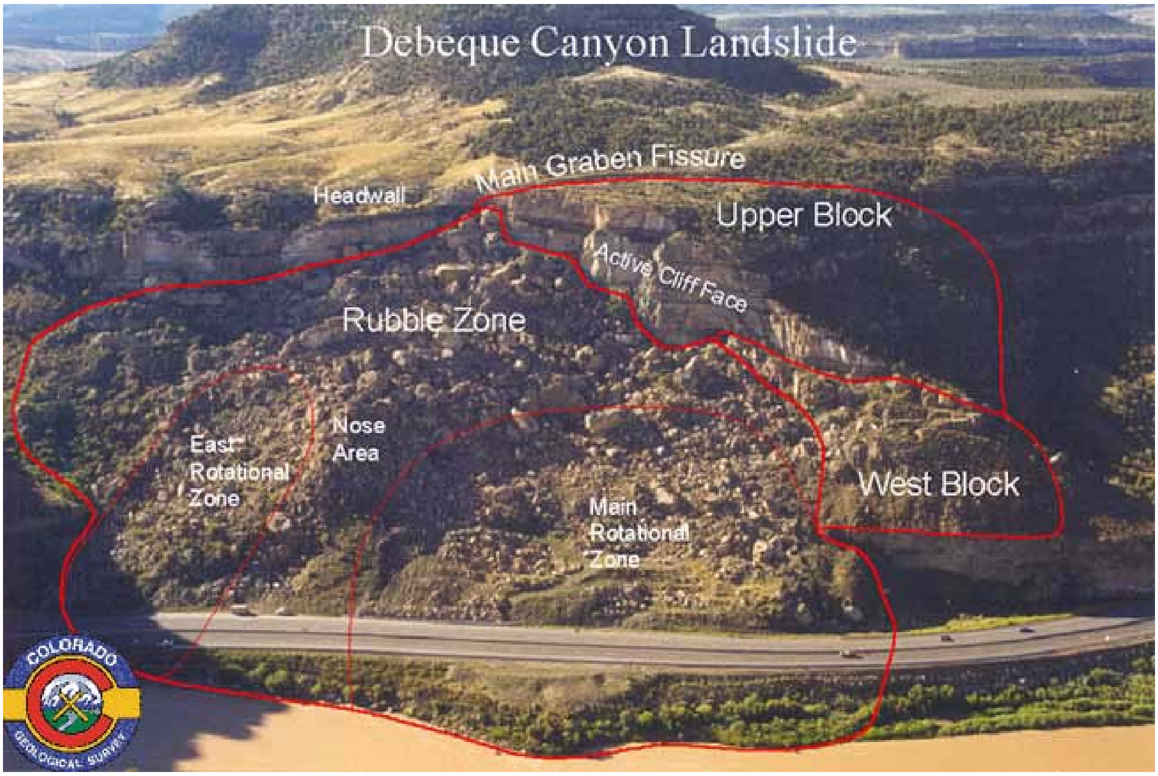

Figure 30. View to the southeast showing delineated morphologic divisions of the DeBeque Canyon Landslide (Photo by J. L. White, Colorado Geological Survey).

It appears that the landslide was initiated during the Late Pleistocene, when downward erosion of the Colorado River exposed thick, weak shale beds that later failed, causing fissuring along pre-existing shear zones and prominent joints in the overlying sandstones (White and Higgins, 2000; White, 2003). Large blocks from these failures add bulk to the central rubble zone and these loads cause it to perpetually creep, loading rotational failures at the base of the rubble zone. Although the landslide appears to be constantly active, a catastrophic failure appears unlikely at this time. The landslide is currently monitored with a variety of instruments that are maintained by Colorado Geological Survey geologists with assistance from the U. S. Geological Survey.

Rockfalls

Rockfalls are a deadly hazard, killing a few travelers along Colorado roadways almost every year, in spite of considerable efforts to evaluate and mitigate these hazards. Colorado engineering geologists have played a major role in advancing the understanding of rockfall phenomena. They have provided improved computer models that simulate rockfalls, and have developed, tested, and installed a variety of mitigation devices. The Glenwood Canyon project, described earlier, provided a major incentive for research on rockfall and the development of mitigation procedures. The Colorado Rockfall Simulation Program (CRSP) was developed in response to the Glenwood Canyon project requirements, and has since become a widely adopted analysis tool (Pfeiffer, 1989; Pfeiffer and Higgins, 1990; Jones, et al. 2000; Higgins, et al., 2003). Rockfall events occur in many parts of Colorado, and present hazards to residential developments as well as travelers on Colorado highways.

Booth Creek, Vail, Colorado: Serious rockfall hazards exist in the Booth Creek area of the Town of Vail, where ledges of resistant limestone and sandstone form cliffs above residential structures (Figure 31). Following a severe rockfall event in May 1983, the Colorado Geological Survey assisted the Town of Vail in assessing the rockfall hazard at Booth Creek (Stover, 1988a). The Town and property owners in Vail Village Filing 12 formed a Geologic Hazard Abatement District (GHAD). The district has mitigated much of the hazard by constructing a ditch and berm on the slope above the residential area that have effectively caught rocks that continually fall from the cliffs above (Figure 32). The ditch and berm were designed with the aid of computer simulations of rockfalls by CRSP (Pfeiffer, 1989; Pfeiffer and Higgins, 1990; Jones, et al. 2000).

However, on March 26, 1997, another very serious rockfall occurred within the GHAD, both above and to the west of the protection envelope provided by the ditch and berm. The ditch and berm stopped all rocks that fell in that direction, as predicted, but rocks falling outside that protection substantially damaged the Booth Falls Condominiums (Colorado Geological Survey, 1998). Blocks as large as 20 ft x 8 ft x 8 ft detached and toppled from the upper ledge. As they fell, they broke apart and loosened additional rock blocks from outcrops below. Rock fragments randomly fanned out to form a swath more than 500 ft wide in the valley below.

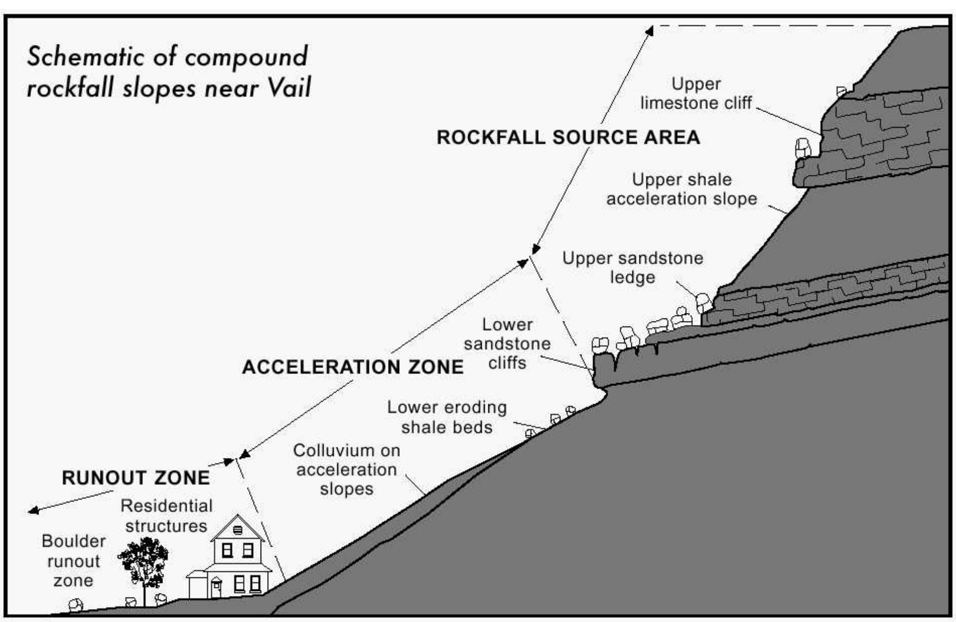

Figure 31. Rockfall Hazard Setting at Vail Colorado (Colorado Geological Survey, 1998).

Castle Rock: In 1981, residents of Castle Rock awoke to earthquake-like rumbles caused by a large block of sandstone that had broken loose from its ledge and was slowly slipping down the slope. It endangered homes below and the Colorado Geological Survey engineering geologists were called upon to investigate. A previously prepared geologic hazards map of Douglas County, prepared by the CGS, had warned about this potential danger (Soule, 1978).

Mitigation efforts were complicated by the ownership of the property: the homes below the cliff were within city limits, while the cliff ledge was county-owned. The Archdiocese of Denver owned the land above the ledge, where runoff from a parking lot may have contributed to the rock slippage. This drama of removing the unstable rock with highly vulnerable homes below received front-page coverage in the local newspapers, and was shown on the NBC Today Show.

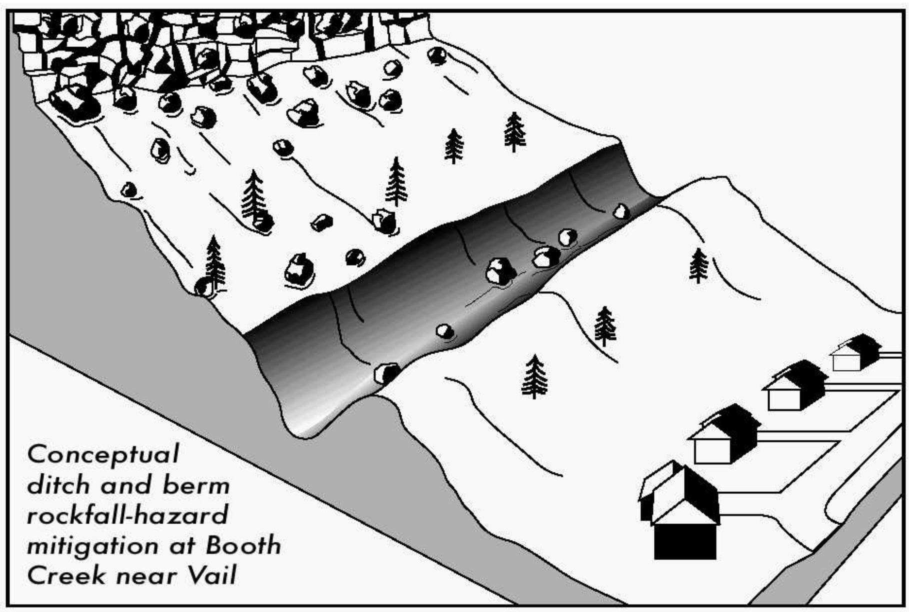

Figure 32. Berm and Bench as Rockfall Protection (Colorado Geological Survey, 1998).

Glenwood Canyon and Colorado Rockfall Simulation Program (CRSP): The requirements for evaluating rockfall hazards within Glenwood Canyon became evident as construction proceeded and environmental and economic considerations made it difficult or impossible to undertake the usual rock blasting and scaling procedures. Research conducted by the Colorado School of Mines, with support from the Colorado Department of Transportation, led to the development of the Colorado Rockfall Simulation Program, or "CRSP" (Pfeiffer, 1989; Pfeiffer and Higgins, 1990; Jones, et al. 2000). CRSP was calibrated on the basis of field experiments conducted by personnel from the Colorado Department of Transportation and Colorado School of Mines. It has been widely used in several states and countries to model rockfall behavior and provides a statistical analysis of probable rockfall events at any given site (Higgins, et al., 2003).

Several rockfall catchment fences, attenuation structures, and other mitigation systems have been designed and placed at critical locations in Glenwood Canyon. Environmental permission has been gained for some limited rock scaling and other preventive measures. Monitoring of identified high rockfall hazard areas continues, but the sheer size of Glenwood Canyon makes it impossible to prevent all rockfall events, and rockfalls continue to occur. Fortunately, most do not cause fatalities or major transportation disruptions (Figure 33).

Figure 33. Typical rockfall event in Glenwood Canyon. This occurred on May 8, 2003 (Photo taken May 9, 2003, by Christian Baxter)

Debris Flows

Debris flows commonly occur in late spring due to snowmelt and during summer months following severe thunderstorms. The topography and geology of Colorado combine to make debris flows a potential hazard in many locations throughout Colorado. Fortunately many debris flows occur in unpopulated areas, or are small enough, so that they do not pose a serious hazard. But in a number of locations, debris flows pose a serious hazard to life and property, and these locations have been, and continue to be, the subject of engineering geological investigations.

In the early 1970's the Colorado Geological Survey investigated geologic problems associated with the proposed Marble ski area in Gunnison County. The results of the investigation recommended against the proposed development because it would occupy large tracts of a narrow, glacial valley that was plagued with debris flows, landslides and potentially unstable slopes, flooding, and avalanches (Rogers and Rold, 1972). Subsequently, a similar report was produced for the Crested Butte-Gunnison Area (Soule, 1976). It describes a number of hazards, including debris flows, and the design of hazards maps and mapping techniques.

A decade later, the Colorado Geological Survey undertook a similar hazards study for the towns of Ouray (Jochim, 1986) and Telluride (Stover and Cannon 1987c). Both towns are located in the San Juan Mountains of Southwestern Colorado, within relatively narrow canyons with steep valley walls, and have had a long history of destructive debris flows. These reports defined the local geology and hydrology, described past events and the resulting damage, and contained a map of hazard zones.

Glenwood Springs, Colorado: The community of Glenwood Springs, situated in valleys at the confluence of the Roaring Fork and Colorado rivers, has long been aware of the debris flow hazard, with the first recorded debris event occurring in 1903, but formal investigations did not begin until the early 1970's (Mears, 1977; Jochim, et al., 1988). More than 20 tributaries enter the main valleys, and most are potential debris flow hazard zones (Figure 34). More than 18 debris flows were recorded between 1903 and 1985, with well-documented events in 1977, 1981, 1984, and 1985. Thus Glenwood Springs became the focus of several geohazard studies sponsored by the Colorado Geological Survey in the 1970's and 1980's (Mears, 1977; Lincoln-Devore, 1978; Soule and Stover, 1985). Glenwood Springs was a case study in the 1988 state landslide mitigation plan (Jochim, et al, 1988).

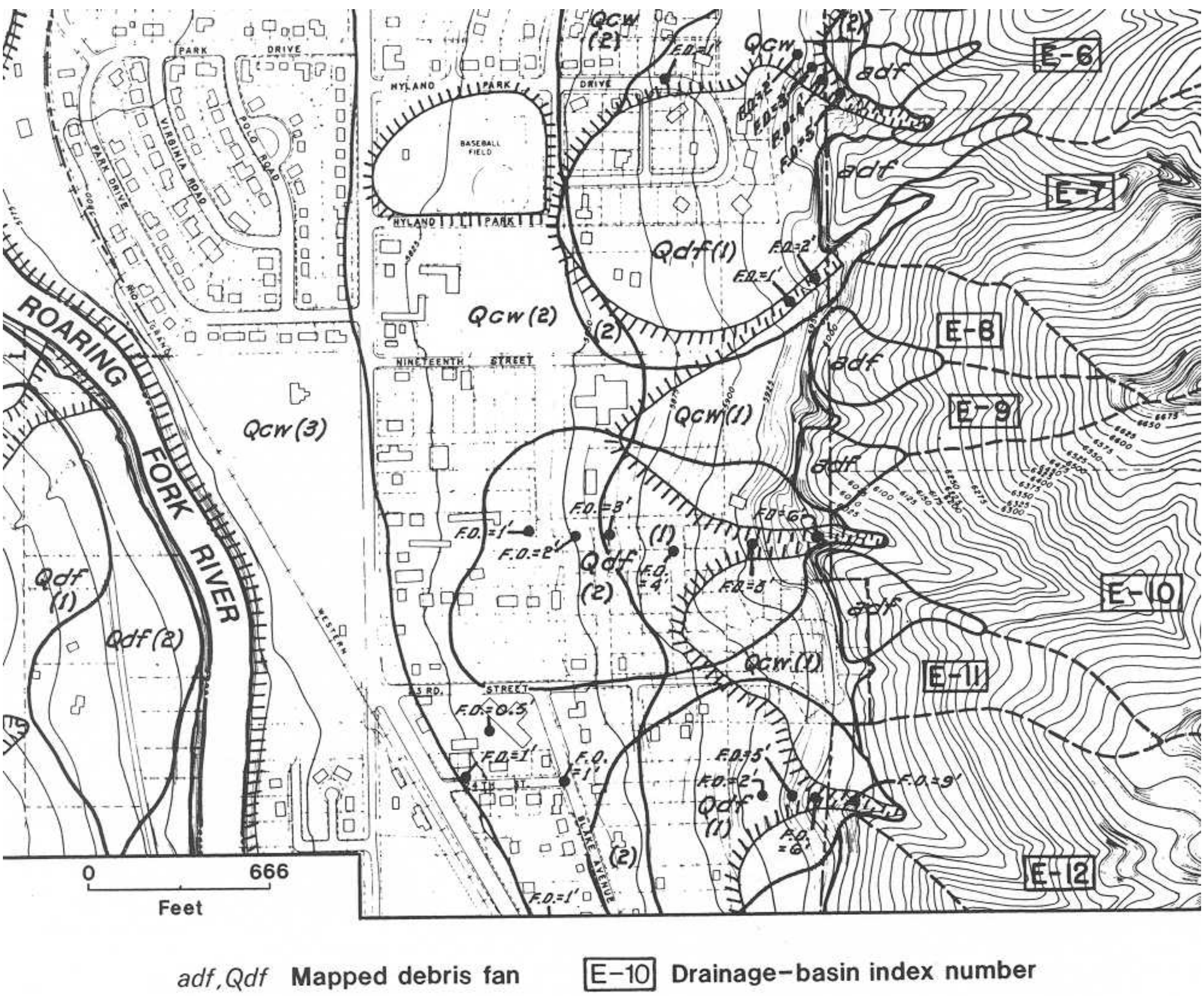

Figure 34. Detail of map showing debris fans at Glenwood Springs (Lincoln-Devore, 1978).

Impacts of Wildfires: In July 1994, a disastrous wildfire occurred on Storm King Mountain adjacent to Glenwood Springs. Subsequently, rains caused debris flows that blocked Interstate 70 (Figure 35). These events caused renewed scientific interest in the relationships between wildfires and debris flows (Cannon, et al., 1995; Kirkham, et al., 1996; Kirkham, et al., 2000; Cannon, et al., 2001). Dry weather conditions in subsequent years resulted in significant additional wildfires near Glenwood Springs, and also at many other locations in Colorado, which added to the urgency of on-going investigations (Colorado Water Conservation Board, 1996; Henz Meteorological Services, 1998; Cannon, 2001; Cannon, et al., 2002)

Figure 35. Cleaning debris-flow material from an off-ramp on Interstate-70 near Glenwood Springs, Colorado, September, 1994 (Photo by Lynn Highland, U.S. Geological Survey).

Debris Flows along the I-70 Corridor West of Denver: The mountainous portion of Interstate-70 is susceptible to a variety of hazardous slope movements. Beginning in 1997, the U.S. Geological Survey, in cooperation with the Colorado Geological Survey and the Colorado Department of Transportation, installed and began to monitor benchmarks on unstable and potentially unstable slopes along the I-70 corridor between Golden and the Eisenhower Tunnel. These monitoring activities expanded to include broader hazard investigations by engineering geologists employed by both the Colorado Geological Survey (Soule, 1975; Colorado Geological Survey, 1999; Soule, 1999; Widmann and Rogers, 2003) and by the U.S. Geological Survey (Savage, et al., 1998; Coe, et al., 2002; Godt and Coe, 2003). These studies have identified and evaluated a series of locations where landslides, rockfalls, and debris flows may disrupt the highway or pose a threat to private residences or commercial properties.

The debris flow hazard was clearly demonstrated on July 28,1999. About 480 debris flows were triggered by an afternoon thunderstorm along the Continental Divide in Clear Creek and Summit counties, with several of these debris flows affecting Interstate 70, U.S. Highway 6,and the Arapahoe Basin ski area. One flow initiated on the south flank of Mount Parnassus, traveled about 1.5 mi down Watrous Gulch and deposited almost 30,000 cubic yards of bouldery debris on Interstate 70 (Figure 36). The Interstate was closed to traffic for about 25 hours. Fortunately no injuries or fatalities resulted from any of the debris flows. Additional information on the debris-flow event, and the damage that resulted, is available in a field-trip guidebook published for the Geological Society of America 2002 annual meeting in Denver, Colorado (Coe, et al., 2002).

Figure 36. Clean-up in progress on July 29, 1999, of debris flow deposits at Watrous Gulch on Interstate 70 about 2 mi west of Barkerville. Note Vehicles for scale (Photo by Ed Harp, U. S. Geological Survey).

Floods

While floods are a widespread hazard throughout Colorado, the Colorado Front Range is subject to some of the most intense rainfall events in the United States and these inevitably cause serious floods. Table 3 documents some of the largest rainstorms during the 20th century along the Front Range of Colorado. The list includes all known storms in this region that exceeded 10 in of rainfall. Most of these storms had associated fatalities. Two floods resulting from dam failures have been discussed previously.

The Denver metropolitan area has experienced a series of floods, averaging at least one major flood each decade, beginning in 1844, as documented in Table 4 (Costa and Bilodeau, 1982). Floods have caused comparatively few deaths, but property damage has been large. The most disastrous flood occurred on June 16, 1965, and caused over $500 million in damages and the loss of six lives (Mattai, 1969). Another flood in May 1973 resulted in $50 million in damages (Hansen, 1973). As a result of these floods, the U.S. Army Corps of Engineers was authorized to construct flood control reservoirs on the major drainages threatening the Denver metropolitan area. Cherry Creek Reservoir on Cherry Creek was completed in 1950, Chatfield Reservoir on

Table 3: Large Local Storms of the 20th Century along the Colorado Front Range (Data from McKee and Doesken, 1997).

Storm Name	Date	Maximum Rainfall
Livermore/Boxelder	May 20-21, 1904	8”
Pueblo/Penrose	June 2-6, 1921	6-12”
Cherry Creek/Hale	May 30-31, 1935	12-24”
Northern Front Range	September 2-3, 1938	6-10”
Rye (South Front Range)	May 18-20,1955	6-13”
Plum Creek (and others)	June 16-17,1965	14-16”
Big Elk Meadows	May 4-8, 1969	6-14”
Big Thompson Canyon	July 31, 1976	12”
Frijole Creek	July 2-3, 1981	8-16”
Fort Collins/Spring Creek	July 27-28, 1997	14.5”
Pawnee Creek	July 29-30, 1997	15.1”

Table 4: Major Floods Affecting the Denver Metropolitan Area Prior to 1982 (From Costa and Bilodeau, 1982).

Date	Stream	Peak Discharge (cfs)	Note
1844	South Platte	unknown	Earliest historical flood
May 1864	Cherry Creek	20,000?	19 killed; all bridges across Cherry Creek destroyed
June 1864	South Platte	unknown	Heavy rain on snow in upper basin
May 1867	South Platte	unknown	Greater than 1864 flood
May 1876	Cherry Creek and South Platte	11,000?	Great destruction
May 1878	Cherry Creek	unknown	Less than 1864 flood; all bridges across Cherry Creek destroyed
May 1885	Cherry Creek	20,000	Large historical flood
June 1894	South Platte	14,000	-----
July 1912	Cherry Creek	11,000-15,000	Over $500,000 damages in Denver
June 1921	South Platte	8,790	500 homes inundated in Denver
Aug 1933	Cherry Creek	16,000	Castlewood Dam failure; $800,000 damages in Denver
Sept l933	South Platte	22,000	------
June 1965	South Platte	40,300	Six drowned; $300 million in Denver
May 1973	South Platte	18,500	$50 million damages; 1.1 times 50-yr flood

the South Platte was completed in 1976, and Bear Creek Reservoir west of Denver was completed in 1976 (Costa and Bilodeau, 1982). Only Clear Creek remains unregulated.

Engineering geologists were involved in the site investigations of these dams, and during their design and construction. In addition, engineering geologists have been active in evaluating and responding to these flood hazards. For example, within a few days of the Big Thompson flood (described below), several engineering geologists working for the Colorado Geological Survey were in the Big Thompson Canyon studying the storm impact on hill slopes and drainage basins. Over a two-month period, they worked full-time on this project to document what occurred geologically and why. Their results were presented to the Colorado legislators in the falloff 1976 and were subsequently published as a Colorado Geological Survey Environmental Geology report (Soule, et al, 1976). One result of these studies was a program of installing public warning signs in hazardous mountain canyons in Colorado (Figure 37).

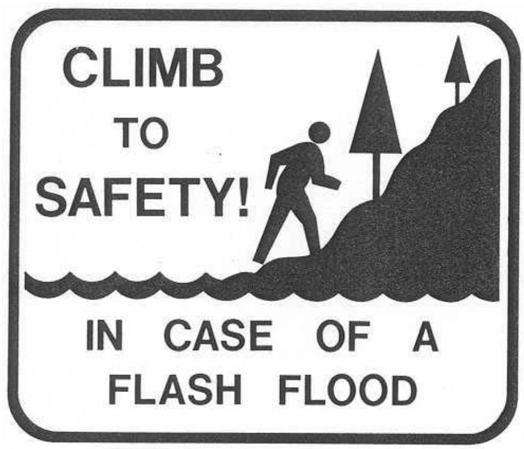

Figure 37. Sign placed in Colorado mountain canyons subjected to flash flooding hazards.

Big Thompson Canyon: On the pleasant summer afternoon of July 31, 1976, several thousand people enjoyed hiking, fishing and relaxing at their campsites in the Big Thompson Canyon. As the afternoon wore on, a thunderstorm developed, accompanied by heavy rain. Unlike most such storms, the storm remained stationary and did not drift east. Heavy rain continued to fall; eight in of rain fell in one hour. The result was a massive debris and flood event – a rampaging wall of water and debris traveled rapidly through Big Thompson Canyon (McCain, et al., 1979; Soule, et al., 1976). Within two hours the Big Thompson Canyon Flood created over $30 million of property damage and killed at least 139 people (Figure 38). This event was perhaps the worst natural disaster to ever occur in Colorado. Subsequent studies concluded it was a 10,000-year event.

Fort Collins Flood: On the evening of July 28, 1997, an extreme flood disaster occurred in Fort Collins after a storm produced the heaviest rains ever documented to have fallen over an urbanized area in Colorado during recorded history. The storm occurred in stages, and dropped 10 to 14 inches in 31 hours in a large area around Fort Collins. The heaviest hourly precipitation occurred at the storm's end, which is different from most storms, and may have exacerbated the flooding (McKee, 1997). Runoff was dramatic and some peak discharges greatly exceeded

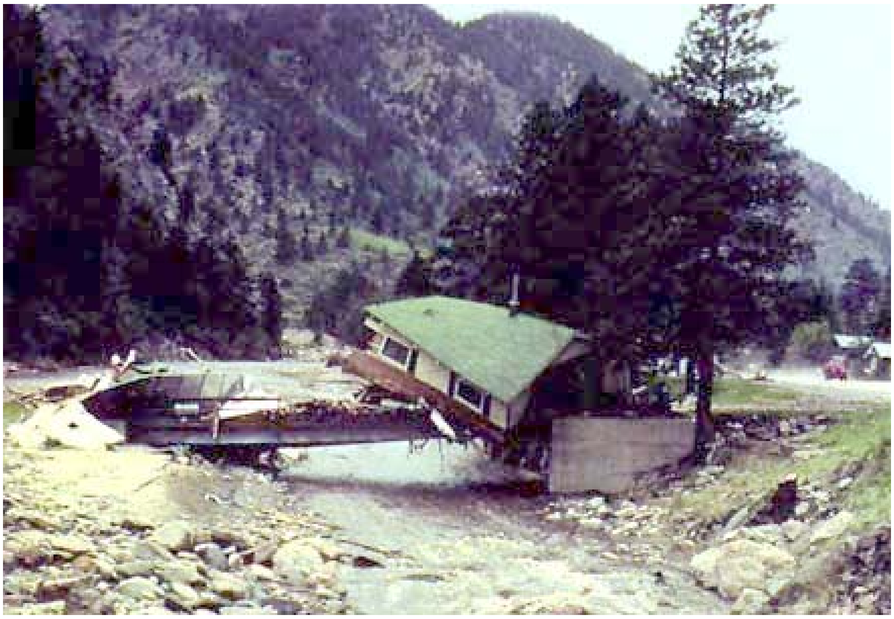

Figure 38. A cabin lodged on a private bridge just below Drake, Colorado (Photo by W. R. Hansen, USGS on 8/13/76).

projected 100-year and 500-year flows. The City Manager's report showed five people dead, 54 people injured, loss of about 200 homes, and 1500 homes and businesses damaged throughout the City (Figure 39). Damages at Colorado State University were unusually severe, totaling in the range of $100 million, including building damages, about 425,000 library volumes inundated, and loss of a semester's textbooks in the bookstore [see http://www.geocities.com/Pipeline/Reef/6415/fcollinslink.html].

Figure 39. High water and debris along Spring Creek (photo courtesy City of Fort Collins).

Problematic Soils

Colorado has several difficult foundation conditions that are encountered in many parts of the state. These problems keep many engineering geologists fully occupied. In recent decades, population growth and recreational developments have increasingly encroached on areas underlain by problematic soils and bedrock. These problems have developed in two distinct geographic centers.

On the western slope soils subject to hydrocompaction are found in close proximity to areas underlain by evaporate deposits and thus subject to dissolution and karst phenomena. Both of these lead to severe foundation distress and potential collapse of buildings and roads. At some locations, both conditions may exist. Unfortunately these hazards are located along and adjacent to the Interstate 70 corridor and in the lower Roaring Fork valley where development pressures are at their greatest.

Along the Front Range swelling soils and bedrock formations underlie many areas undergoing residential development. Because the bedrock formations are steeply dipping in some parts of the area, traditional engineering solutions to foundation design do not apply. Considerable damage occurred before the problem was identified and engineering geological investigations were undertaken to identify the areas with high risk, and to develop suitable mitigation procedures.

Hydrocompacting Soils and Karst Hazards on the Western Slope: The existence of such soils was known for many years. Around 1910, when the earliest settlers began to apply irrigation to develop fruit orchards in the Colorado and Uncompahgre valleys, they discovered that the land surface sometimes collapsed as much as four ft when initially wetted. Such settlement cause problems as the irrigation canals lost proper elevations and some orchards became waterlogged (Colorado Geological Survey, 2001).

In the mid-1970's, engineers building western portions of Interstate 70 encountered hydrocompactible soils (Shelton, et al, 1977). This new highway construction and population growth spurred a renewed interest in the hydrocompaction phenomenon (Beckwith and Hansen, 1989; Colorado Geological Survey, 2001). These investigations determined that hydrocompactible soils occur in areas that have less than 20 in of annual precipitation and are the products of recent and rapid deposition, which creates an inherently unstable internal structure. Following the Colorado River, Interstate 70 encountered many miles of hydrocompactible loess and alluvial or debris fan deposits in the geological environment shown in Figure 40. Following engineering geological investigations, the susceptible areas were subjected to pre-wetting prior to construction.

In the same general area, especially in the Roaring Fork valley between Carbondale and Glenwood Springs and the Eagle River valley between Edwards and Gypsum, additional collapsing problems were encountered. In this case karst phenomena resulted from the presence of extensive evaporate deposits at shallow depths. These hazards threatened rapidly developing residential areas as well as highways. They were the subject of several engineering geological studies that resulted in reports written to guide landowners, planners, municipal and county land-use regulators, and the geotechnical and civil engineering community on the formulation of appropriate and proper types of investigation (Soule and Stover, 1985; Colorado Geological Survey, 2001; White, 2002).

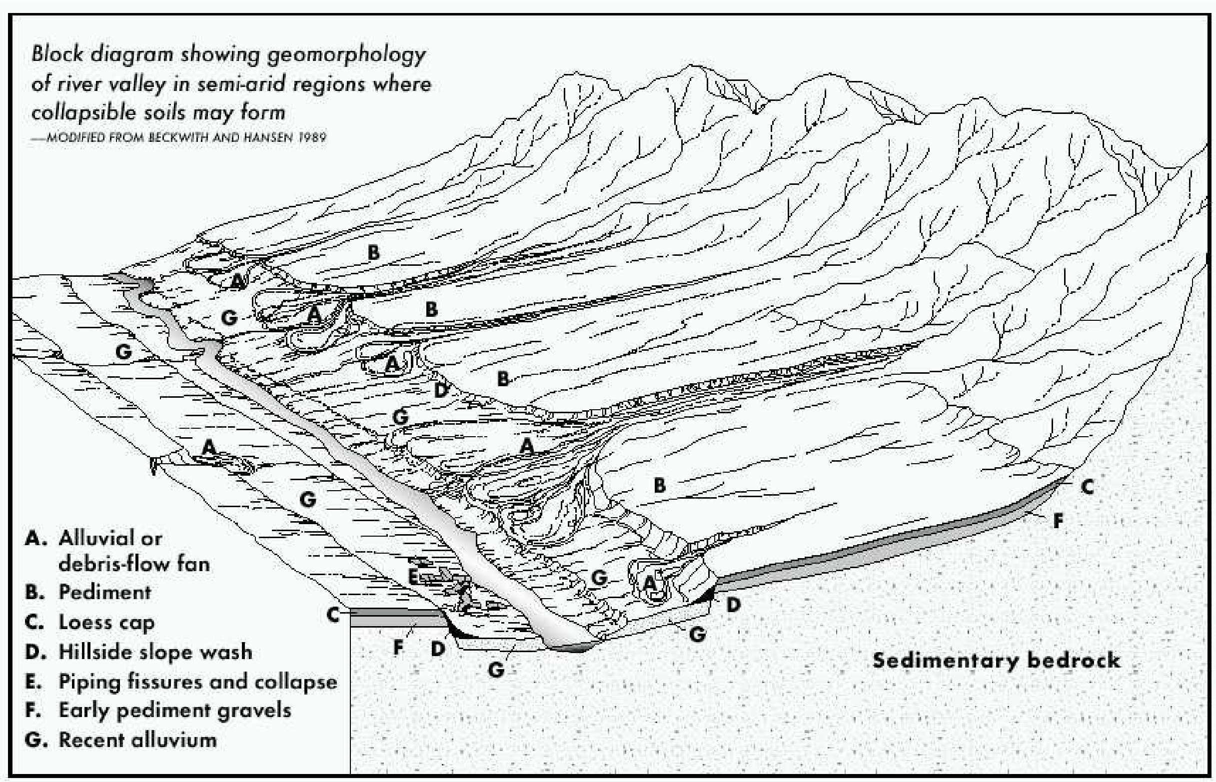

Figure 40. The geological setting producing hydrocompactible soils along Interstate 70 in western Colorado (Colorado Geological Survey, 2001; modified from Beckwith and Hansen, 1989).

Swelling Soils and Heaving Bedrock in the Front Range: Swelling soils are encountered in most parts of Colorado outside the high mountains, however the population concentration along the Front Range Urban Corridor results in the majority of the damage occurring there. Shrinking and swelling of montmorillonitic clay soils as they undergo moisture changes is a well-known and widely distributed engineering geological problem.

However, at certain locations along the Front Range, geological conditions conspire to make the problem even more severe and difficult to resolve, because accepted engineering designs for foundations, such as floating slabs or pier-and-grade-beam designs, do not work reliably where steeply-dipping swelling claystone bedrock formations are found. Engineering geologists have investigated these problems since the early 1970's when new residential construction began to expand into susceptible areas (Hart, 1974). In the 1990's large residential developments began in many susceptible areas and the rates of damage increased markedly (Figure 41). Engineering geologists at the Colorado Geological Survey undertook extensive investigations to map and define the extent of the high-risk "heaving bedrock" areas (Himmelreich and Noe, 1999; Noe and Dodson, 1999; Berry, et al., 2002), and to evaluate the phenomenon (Noe, 1997; Noe and Dodson, 1999; Berry, et al., 2002).

Figure 41. Surface distortions caused by heaving bedrock, Jefferson County (Photo by D. Noe, Colorado Geological Survey).

These activities were supplemented with an extensive outreach program to assist homeowners in reducing damage caused by swelling soils. A critically acclaimed "homeowners guide" was developed (Noe, et al., 1997). This publication is distributed to home- buyers in affected areas for disclosure purposes, and over 100,000 copies have been distributed since 1997. It has received the John C. Frye Award in Environmental Geology by the Geological Society of America in 1998, and the Edward B. Burwell, Jr. Memorial Award of the Engineering Geology Division of the Geological Society of America in 2001. This publication represents an important milestone in the public's awareness and acceptance of the importance of engineering geology in Colorado.

Subsidence Above Inactive Coal Mines: Abandoned mines are a hazard in many parts of Colorado; all the important mining districts are underlain by numerous old workings that pose hazards to the unwary. Drainage from abandoned workings often produces highly mineralized and acidic water that causes water quality issues. These topics have been the subject of ongoing engineering geology studies for many years (Amuedo and Ivey, Inc., 1975; Hynes, 1987; Matheson, 1987; Padgett, 1987; Hatton and Turney, 1989; Colorado Geological Survey, 2001).

However, several rapidly urbanizing areas along the Front Range have encountered a much more serious hazard. They have expanded over old coal mine workings. These coal mines used room and pillar methods at comparatively shallow depths, so that collapse of the workings produces noticeable surface failures. The problem was first investigated in the mid- 1970's as housing developments encroached on the abandoned workings of the Boulder-Weld Coalfield (Amuedo and Ivey, Inc., 1975). In the early 1980's, it was discovered that an entire subdivision in the suburbs southwest of Denver had been constructed over an old coal mine. Investigations for a new shopping center revealed the mine, yet the county planning office had no records showing the mine. Subsequently, developments in Colorado Springs also encountered subsidence from inactive coal mines. In the late 1980's the Colorado Geological Survey undertook additional abandoned mine studies north of Denver in Weld County around the towns of Firestone, Frederick, and Dacono (Hynes, 1987). The Colorado Geological Survey published an annotated bibliography that documents subsidence investigations in five Front Range counties (Hatton and Turney, 1989). The Colorado Geological Survey continues to maintain an up-to-date coal mine subsidence library that provides information on historic coal mine and subsidence risk investigations and assessments (Colorado Geological Survey, 2001). New investigations are conducted as new collapse events occur (Figure 42).

Figure 42. Investigating a coal mine shaft collapse in a trailer park. (Photo by Colorado Geological Survey).

Earthquakes

Colorado was long considered an area of low seismicity, with only a minor potential for experiencing damaging earthquakes in the future (Algermissen, 1969). Although Colorado has a history of strong earthquakes in the recent geologic past, there has been much less research effort directed toward understanding its earthquake hazard than in the neighboring states of Utah and New Mexico (Colorado Geological Survey, 2002a). Only recently has the heavily populated Front Range been officially recognized as having the potential for earthquakes that may be characterized as “low frequency and high consequence” (Colorado Office of Emergency Management, 1999).

Re-assessment of Colorado earthquake hazards began with studies by the U.S. Geological Survey (Scott, 1970). By the early 1980's, extensive environmental impact valuations for dams and similar works identified many active faults that might yield earthquakes (Kirkham and

Rogers, 1981). The Colorado Geological Survey continued research into historical earthquake events (Oaks and Kirkham, 1986; Rogers and Kirkham, 1986). These studies revealed that the strongest earthquake recorded in Colorado occurred in 1882. It had a magnitude of 6.6 and was centered about 10 mi north of Estes Park (Kirkham and Rogers, 1986; Spence, et al., 1996). Recent geologic deposits provide evidence of pre-historic earthquakes with magnitudes 7.0 or higher.

Colorado is famous for earthquakes induced or triggered by humans. In September 1961 the U.S. Army drilled a two-mi deep injection well at the Rocky Mountain Arsenal, northeast of Denver in order to dispose of some highly toxic liquids. Injection began in March 1962, and within less than a year, thousands of small earthquakes were recorded near the Arsenal. In 1967, two earthquakes of about Magnitude 5.0 occurred, and the largest earthquake (magnitude 5.3) caused $1-million damage to Commerce City and north Denver. In November 1965, David Evans, a Denver-based consulting geologist, publicly suggested that these earthquakes were the direct result of the Army's fluid injection operations (Evans, 1966). Evans supported his conclusions with an analysis of temporal correlations between fluid injection operations and earthquakes (Bardwell, 1966). Figure 43 graphically demonstrates the temporal correlation. The Army denied any correlation, and many geologists doubted it, so several studies were undertaken (Healy, et al., 1968; Hollister and Weimer, 1968; Scopel, 1970; Hermann, et al., 1981). Although a direct cause and effect could not be proven, fluid injection was stopped in February 1966. Earthquakes, however, continued to occur for a year or more. This was explained by analyses described by Healy et al. (1968).

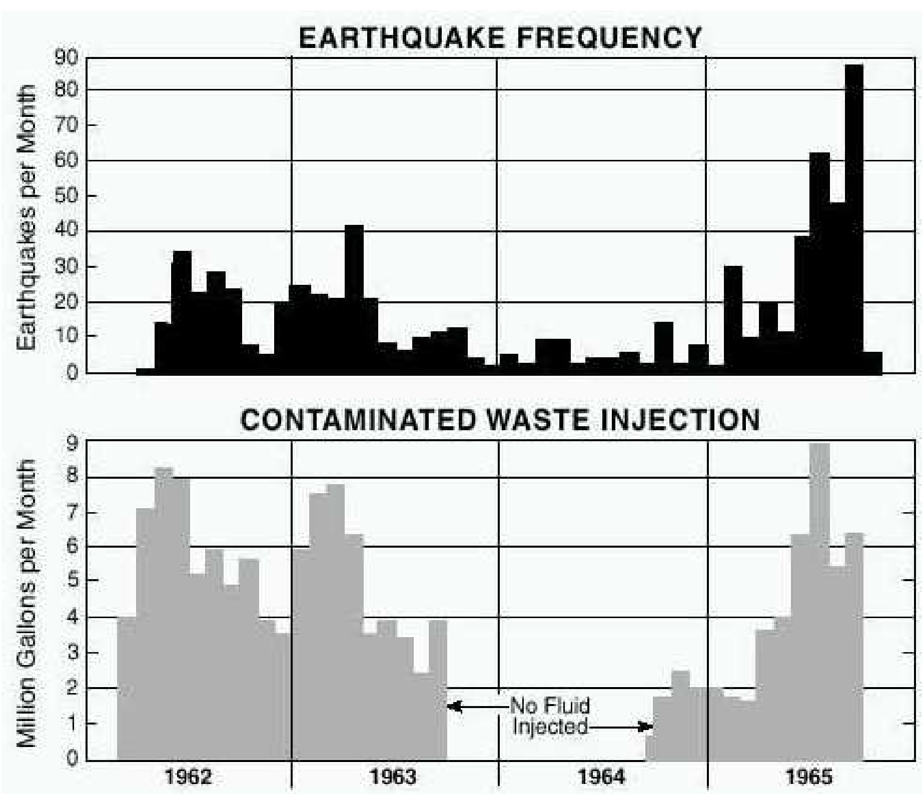

Figure 43. Charts showing the temporal correlation between Rocky Mountain Arsenal Well injection volumes and earthquake activity (after Evans, 1966).

Between 1968 and 1970 the U. S. Geological Survey undertook additional experiments on whether water injection could induce earthquakes. They monitored water flooding operations at the Rangely oil field in northwestern Colorado. This field experienced minor earthquakes (up to 50 per day) when water flooding was underway, but the rate dropped to less than 10 per day when injection ceased (Munson, 1970; Colorado Geological Survey, 2002a). More recently, minor earthquakes have been triggered by injection of highly saline water by the U. S. Bureau of Reclamation in the Paradox Valley in western Colorado. In an attempt to reduce the amount of salt being introduced by the Dolores River into the Colorado River, the injection of brine began in 1995. Since that time over 3,000 minor earthquakes have been recorded. After a Magnitude 4.3 earthquake in May 2000, the rate of injection has been reduced and so far no similar earthquakes have occurred (Colorado Geological Survey, 2002a).

Earthquake hazards continue to be the subject of investigations (Widmann, et al., 1999; Kirkham and Rogers, 2000). In August and September of 2001, a swarm of earthquakes, including two with Magnitudes of 4.0 and 4.6, caused minor damage to homes and businses near Trinidad in southern Colorado (Colorado Geological Survey, 2002a). The U.S. Geological Survey rapidly deployed a dense network of portable, temporary seismographs to study the earthquakes (Meremonte, et al, 2002). Earthquakes had occurred previously in the same area in 1966 and 1973, but some people wondered if the 2001 earthquakes were related to water injection activities connected with coal-bed methane production. No definitive relationship has been determined.

ENGINEERING GEOLOGY IN RESPONSE TO SOCIETAL CONCERNS

In 1943, a reorganization of the U.S. Bureau of Reclamation brought considerable numbers of Bureau engineers and geologists to Denver to staff the Engineering and Research Center (now named Reclamation Service Center) located at the Denver Federal Center. Bureau civil engineers and engineering geologists were responsible for the design and construction of water supply projects in Colorado and throughout the western United States. Their work has been an important component in the history of engineering geology activity in Colorado (Simonds, 1998). After World War II, Denver became headquarters and one of the major centers for a new section, and later a Branch of Engineering Geology, established by the U.S. Geological Survey. This nucleus of talented professionals stationed in Colorado provided the spark that encouraged the spread of engineering geology practice in the Colorado region. The experiences and missions of these two groups of Federal employees were largely complementary, and this added to the richness of the practice of engineering geology in Colorado. During this same period, the Denver Water Board employed staff engineering geologists and consultants as they made plans to develop and deliver increasing volumes of water to municipalities of the Denver Metropolitan region.

The 1967 report by the U.S. Geological Survey on the engineering geological investigations at the new U.S. Air Force Academy site in Colorado Springs (Varnes and Scott, 1967) set a high standard for engineering geology site investigations. The report devotes only about 30% of its volume to describing the setting and general geology, while about 60% of the report deals with engineering geology issues and 10% was devoted to a separate evaluation of ground water (Cardwell and Jenkins, 1967). The Academy site was complex, and was subdivided into several areas. The report describes the engineering geology conditions for each area separately. Within each of these sections, some 10% - 15% of the section is devoted to introductory issues, while 45% - 75% is devoted to field and laboratory investigations, and 20% - 40% provides recommendations for design. This report influenced the standard of practice expected in subsequent engineering geological investigations in Colorado and elsewhere.

During the 1960's, the practice of engineering geology spread to Colorado state agencies faced with problems related to greatly expanded transportation system and residential development needs. A Geotechnical Section was established in the Colorado Department of Highways (now Colorado Department of Transportation) and a new Colorado Geological Survey was created after a dormant period of several decades. The reborn Colorado Geological Survey included an Engineering Geology Section that moved aggressively to deal with the State's land use, geologic hazard and geology-related environmental issues, problems and information needs. In the higher education scene, some of the major state academic institutions – Colorado School of Mines, Colorado State University, and the University of Colorado – developed undergraduate and graduate research and educational programs that provided many geology graduates with Colorado "roots."

Also in these formative years, the private sector construction and engineering interests increasingly realized that Colorado's complex and often hazardous geological environment required pertinent geologic input in most project work. This included planning, design, and development phases of residential, infrastructure, and natural resource projects. As a result, a strong and skilled geotechnical consulting community emerged in Colorado. Interactions among geoscientists from these various quarters through professional meetings and societies and shared projects was conducive to technology transfer, and this led to many creative advances and contributions in the practice of engineering geology in Colorado.

In the 1970's, increased emphasis on environmental values materially changed and enlarged the role and scope of engineering geology. In Colorado, this led to two important and more or less contemporaneous developments – the establishment of the "Front Range Urban Corridor Project" by the U.S. Geological Survey (Hansen, et al., 1982), and a series of key land-use regulations passed by the Colorado Legislature in the 1970's that required the Colorado Geological Survey to review development proposals for geologic suitability in order to protect prospective homeowners and their property. In response to these mandates, the Colorado Geological Survey produced a series of public information reports during the 1970's dealing with a variety of engineering geology and geological hazard topics (Rogers and Rold, 1972; Hart, 1974; Rogers, et al., 1974; Soule, 1975; 1978; Junge, 1978a; 1978b; Shelton and Prouty, 1979).

During the early 1970's the U.S. Geological Survey undertook a series of innovative geological assessments in a corridor along the Front Range stretching from north of Fort Collins to south of Colorado Springs, while the Colorado Geological Survey undertook a number of investigations in other parts of the state. The references provided with this paper include many of the critical studies of these two agencies. The Front Range Urban Corridor Project expedited the completion of basic geological mapping along the Front Range; it also encouraged innovative and experimental maps. The basic data were ultimately re-compiled onto a series of thematic maps at a scale of 1:100,000. Three sheets were used to cover the Front Range region, a northern sheet ("Boulder – Fort Collins – Greeley"), a central sheet ("Greater Denver"), and a southern sheet ("Colorado Springs – Castle Rock"). The maps were published in the U.S. Geological Survey's Miscellaneous Investigation Series with consistent numbers, but unfortunately the themes were not placed in a consistent sequence (Table 5).

Table 5. Regional Thematic Maps Produced by the U.S. Geological Survey's Front Range Urban Corridor Project

Map Theme	Map I-855 Series ("Boulder – Fort Collins – Greeley")	Map I-856 Series ("Greater Denver")	Map I-857 Series ("Colorado Springs – Castle Rock")
Potential Sources of Gravel and Crushed-rock Aggregate	I-855-D	I-856-A	I-857-A
Land Use Classification	I-855-B	I-856-E	I-857-B
Lakes	I-855-A	I-856-B	I-857-E
Flood Prone Areas	I-855-E	I-856-D	I-857-C
Availability of Hydrologic Data	I-855-C	I-856-C	I-857-D
Natural/Historic landmarks	I-855-F	I-855-F	no map
Historical Trails	no map	I-856-G	no map

As part of the Front Range Urban Corridor Project, innovative engineering geology maps were produced for several 7.5-minute quadrangles, including Golden, Highlands Ranch and Morrison (Gardner, et al., 1971; Maberry and Lindvall, 1974; Simpson and Hart, 1980). At the same time some additional experiments were undertaken with the use of computers to store and display Jefferson County map data in connection with Open Space studies – essentially these were very early GIS products (Smedes and Turner, 1975; Turner, 1976a; 1976b). In 1973 the Colorado Legislature mandated an evaluation of aggregate resources along the Front Range, so the Colorado Geological Survey conducted a detailed investigation (Schwochow, et al, 1974; Schowchow and Shroba, 1975), while the U.S. Geological Survey made regional evaluations (Colton and Fitch, 1974; Soule and Fitch, 1974; Trimble and Fitch, 1974a; 1974b).

The 1980's brought new challenges to engineering geology in Colorado. The Colorado Geological Survey lost most of its "general Fund" funding in 1984. The Colorado Legislature required the Colorado Geological Survey to adopt fees for its review activities, and expected it to perform cash- and grant-funded projects for state-, federal-, and local-government agencies. As a consequence the Colorado Geological Survey did secure and complete a large number of projects during the 1980s, including earthquake and seismicity studies, a statewide radon survey, and a study for locating a "Superconducting Supercollider" in the state. Several activities were encouraged by a series of wet years in the mid-1980's that caused a series of landslides throughout the state. These studies led to the production of a statewide landslide hazard mitigation plan (Jochim, et al., 1988; Wold and Jochim, 1989).

Environmental issues became important in many projects. Beginning with the Interstate 70 Vail Pass Project, the Colorado Geological Survey had entered into a partnership with the Colorado Department of Transportation (CDOT) to provide engineering geological investigations and construction support. As the Interstate 70 construction began in Glenwood Canyon, these partnerships grew larger, with Colorado Geological Survey engineering geologists working together with Colorado Department of Transportation geologists and engineers. The Colorado Geological Survey also undertook a variety of environmental and geohazard studies, including an environmental case study in the Windsor area, located between Greeley and Fort Collins with funding and assistance of the U.S. Geological Survey's Urban Front Range Corridor Project (Shelton and Rogers, 1987).

The 1990's were a period of rapid population growth in Colorado. During this period, and continuing to the present time, many geological hazards were revisited and new document produced, often with digital files accompanying traditional maps and text (Creath, 1996; Nuhfer, et al., 1996; Rold and Wright, 1996; Noe, et al, 1997; Johnson and Himmelreich, 1998; Rogers, 2002). Renewed concerns about aggregate availability caused revision and updating of the aggregate assessments of the 1970's (Lindsey, 1997; Cappa, et al., 2000; Wilburn and Langer, 2000). Wild fires became an increasing concern, especially the tendency of burned areas to produce severe debris flows and flooding when subjected to intense precipitation (Cannon, 2001; Cannon, et al, 2001; 2002). In the late 1990's, new partnerships among various levels of government, the private sector, and academia were applied to some complex engineering geology investigations, such as the DeBeque Canyon landslide, which began threatening Interstate 70 in Mesa County (Miller and Higgins, 2000; White, 2003). During the 1990's the Colorado Geological Survey undertook an expanded public outreach and technology transfer role to educate the public and various stakeholders about geologic hazards, including conferences in various locations throughout Colorado (Figure 44).

Figure 44. Part of a flyer calling attention to a geological hazards outreach meeting.

SUMMARY AND CONCLUSION

This paper has traced the history of engineering geology in Colorado according to applications, and also identified important individuals and organizations that have influenced the practice of engineering geology in Colorado. Engineering geology has evolved in response to changing societal concerns and expectations.

The last half of the 20th Century witnessed a rapid expansion of engineering geology in Colorado. Design and development of new infrastructure required by a rapidly growing diverse society placed new demands on engineering geologists. Individuals and organizations made significant innovations and contributions to the practice of engineering geology in Colorado and beyond. Colorado has become an important center for innovations in engineering geology.

ACKNOWLEDGEMENTS

Development of this historical review has relied on information freely contributed by many individuals, government agencies, and private firms. We gratefully acknowledge this assistance from these many sources, which are too numerous to name individually. This report also highlights the important role of many engineering geologists, living and dead, to the quality of life we all experience in Colorado. On behalf of society, we thank all engineering geologists connected with the historical events described in this report for their contributions to the development of Colorado. We thank two anonymous referees for critical review of the draft version of our manuscript.

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C-BT statistics: http://www.ncwcd.org/project&features/cbt_by_the_numbers.asp
Horsetooth Reservoir Modernization: http://www.abouthorsetooth.com

USBR WEBSITES:
History of the Bureau: [http://www.usbr.gov/cultural/saa.htm

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General Information: http://www.usbr.gov/dataweb/html/crsp.html

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THE FUTURE OF ENGINEERING GEOLOGY IN COLORADO

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Key Terms: engineering geology, future, trends, technologies, education, state of the practice

ABSTRACT

Colorado's complex geology challenges engineering geologists to solve a broad range of problems affecting land use and development. Engineering geologists provide an important and valuable service to homeowners, developers and municipalities by helping to protect the health, safety and welfare of Colorado's growing population. Yet there are areas where we can improve our site characterization, hazard identification and mitigation design capabilities. Additionally, there are interdisciplinary situations where our expertise has not been used to its full potential. This paper discusses engineering geology trends for the future and how improving technology, experience and knowledge can help engineering geologists continue to guide responsible land use decisions and provide safe and cost-effective solutions to engineering problems and geological hazards.

INTRODUCTION

There is wide consensus among engineering geologists and geotechnical engineers that our jobs are becoming more difficult – and necessary – as heavy development pressure drives new construction into increasingly problematic areas. Based on estimates from Colorado Department of Local Affairs, Colorado's population will increase by approximately 44 percent from 2002 to 2025. This will result in increased demand for resources and infrastructure related to water, power, transportation and waste disposal. Engineering geologists are uniquely qualified to help guide land use planning and development in a way that safeguards the public from health and safety hazards and costs related to geologic conditions, topography and environmental constraints.

A symposium titled "Visioning the Future of Engineering Geology: Sustainability and Stewardship" was convened at the AEG-AIPG Joint Annual Meeting in September 2002. The goal of the symposium was to discuss how engineering geologists should best continue to use their "special expertise to protect the public health, safety and welfare in an effective and responsible manner," and "find increasing acceptance of the value of their services" (Tepel, 2002). The papers presented at the symposium are compiled on a CD available as AEG Special Publication 14 (Tepel, ed., 2002).

The scope of this paper is more regional than the AEG 2002 symposium, and presents Colorado geoprofessionals' thoughts on our abilities and the methods needed to achieve the following goals:

- Advance the future state of engineering geology practice in Colorado,
- Identify and mitigate specific geologic hazards and address related land use issues in Colorado,
- Improve site characterization abilities through use of emerging technologies and techniques, and
- Improve communication within the field, and collaboration among geologists, engineers, planners, educators and the public.

METHODOLOGY

The thoughts presented in this paper were gathered from responses to a one-page survey sent to geologists and engineers in the Rocky Mountain region in Fall 2002, and through interviews with professionals practicing in the fields of engineering geology and geotechnical engineering. The one-page survey is included as Appendix I. After receiving responses to the questionnaire, longer interviews and conversations were conducted by Colorado Geological Survey (CGS) staff. Interviewees were selected from suggestions made by CGS staff and others. We also interviewed some of the one-page survey respondents to clarify statements and opinions.

We interviewed professionals from industry, academia and public service, practicing in a variety of geographic locations, and possessing a range of experience. Interviewees were subjected to a number of detailed questions on selected topics (education, technology, resources, communications, etc.) Their statements comprise the bulk of this paper.

RESULTS

State of the Practice

Sentiment among professionals regarding the state of the practice of engineering geology in Colorado is that we are, for the most part, on track. Some feel that there are certain groups and areas of practice where performance may not be up to accepted standards, but the majority of practitioners are responsible professionals. The field of engineering geology continues to draw students, particularly as infrastructure becomes more important and development continues.

Engineering geology in the state of Colorado exists as an area of overlap among geologists, geological engineers and geotechnical engineers. That overlap, and misunderstandings about our differing areas of expertise, sometimes create confusion among the general public and organizations that use our services. Several comments alluded to strong competition between geotechnical and geologic engineers and geologists, with more professionals vying for fewer projects. Engineering geologists can help to clarify our roles by acting as advocates of the profession, specifically by distinguishing the unique value of our work products and methods from those produced by other engineering disciplines.

Some feel that the practice of engineering geology in Colorado is more remedial than proactive, and that we need to improve our hazard mitigation abilities. More effort should be put into anticipating how a site's geology will affect engineering, design and construction during the early phases of project development, rather than reacting to events after a project is complete. For example, retention, deflection and channeling structures are examples of mitigation methods that, correctly designed in the early planning stages of a project, could enable development in or near debris flow hazard areas. However, it is common to see mitigation of such a hazard area take place after final platting, and especially after damage has occurred to structures. Geological hazard mitigation will become more important in residential development as real estate costs continue to increase. Likewise, licensure of engineering geologists could become essential as hazard mitigation concepts and design become part of our routine scope of work.

Registration

Registration and licensure of geologists is a hot topic. Many practitioners feel that engineering geologists should be self-regulating, while others feel that because of a few "bad apples" who choose NOT to self-regulate, we need the discipline offered by licensure. Those opposed to licensure feel that it offers limited benefits, is too restrictive and provides little recourse for maintaining professional accountability. Licensure without a grandfather clause would provide the field with more credibility and would prevent potential stagnation, especially if continuing education were a requirement for maintaining ones license.

Although our neighbors Texas and Utah recently began licensing geologists, and Kansas has been issuing licenses to professional geologists since 1999, there is a substantial amount of pessimism that Colorado will enact licensure legislation. Several responses suggested that the practice of geology in Colorado has been static for the past 30 years and that licensure would not have any effect.

It should be noted that in the International Building Code, which is replacing the Uniform Building Code (in Colorado, as of March 2003, 6 counties and 25 municipalities have adopted the IBC), the broad term "registered design professional" is used to describe those who perform work in the following specific areas (ICC, 2000 and Mathieson, 2002):

Sec. 1613	Active faults
Sec. 1615	Site classification related to seismic considerations
Sec. 1802	Foundation and soils investigations
Sec. 1802.4	Exploratory boring
Sec. 1802.5	Soil boring and sampling
Sec. 1805.3.5	Alternate setback and clearance (from crests and toes of slopes)

In the 2003 IBC, a registered design professional is defined simply as "an individual who is registered or licensed to practice their respective design profession as defined by the statutory requirements of the professional registration laws of the state or jurisdiction in which the project is to be constructed." (ICC, 2003) As discussed in "Geologic Hazards Avoidance or Mitigation, A Comprehensive Guide to State Statutes, Land Use Issues, and Professional Practice in Colorado" (Johnson and Himmelreich, 1998), many Colorado statutes, including Senate Bill 35 (1972, C.R.S. § 30-28-133), addressing county subdivision regulations, House Bill 1041 (1974, C.R.S. § 24-65.1-103), concerning areas of state interest, including geologic hazard areas, and

House Bill 1045 (1984, C.R.S. § 22-32-124), pertaining to geologic suitability of proposed school sites and improvements, require the preparation of analyses and reports about geologic and soil conditions. House Bill 1574 (1973, C.R.S. § 34-1-201, -202) requires that reports containing geologic information must be prepared or approved by a professional geologist. However, Colorado statute currently defines a professional geologist only as a person having graduated from an accredited university with a minimum of 30 semester (45 quarter) hours of undergraduate or graduate work in a field of geology and post-graduate training specializing in geology with five years of geological experience. There are no enforcement mechanisms.

In Colorado, without licensure or registration of geologists, the building code can thus be interpreted to allow geologic work to be performed by non-geologists. This poses potential threats to 1) public safety and welfare, and 2) the rights of engineering geologists to practice their profession.

Those in favor of some form of registration feel it would put geologists on more equal footing with professional engineers and provide some accountability, although fear of litigation should not replace true professionalism. Licensure would also reduce the number of people working in the field that are not qualified to do so. Nationwide trends toward licensure indicate that professional standards in Colorado should be revised and updated and should involve some form of registration or licensure.

Communication

To achieve more credibility and avoid continuing conflict over who should be allowed to practice within the gray areas where engineering and geology overlap, we need to be – or at least think like – both engineers *and* geologists. We have improved in our ability to work and communicate with civil and geotechnical engineers, but we need to keep improving.

Early phases of a site investigation consist of geological investigations related to feasibility, and later phases provide data and concepts necessary for engineering design. When geological hazards are identified, engineering geologists tend to be underutilized in attempts to "engineer" a solution to a single issue, rather than look past the site boundaries and assess local and regional geologic processes within the context of an engineered solution.

There is a disconnect between geologists and engineers, despite the fact that it is critically important that we collaborate to avoid disasters such as the failure of Teton Dam. The geologists and engineers working within the Bureau of Reclamation provide a good example of successful cooperation, but it took a major disaster to achieve that level of teamwork. We should be proactive, and we should work to inform and educate a wider audience about geologic risks, hazards and the value of our services, both within and outside the engineering and geologic communities.

Opinions regarding the decline in the state of the practice are reflected in comments like: site characterization is lacking, more subsurface data should be collected to allow better interpretation of the geology and engineering properties of a site. The trend toward less rigorous site investigations may be due to several factors, including 1) a decrease in funds allocated to field work and laboratory testing by geotechnical firms in the effort to keep project bid prices low, 2) lack of awareness on the part of owners that thorough consideration of site hazards and development constraints can reduce long term maintenance, repair and legal costs, and 3) increased willingness on the part of engineering firms and owners to accept poorly understood risks.

Smaller companies in rural areas practice within relatively broad areas of expertise. The professionals working in these areas tend to split their time between field, laboratory and design work as required. Practitioners in rural areas tend to rely on test pits and trench profiles during site investigations rather than drilling. This difference is primarily related to the costs associated with mobilizing a drill rig over long distances to evaluate a relatively small development site.

Most professionals feel that engineering geologists should be more active in providing insight and expertise at the planning level. We need to educate Colorado planners about the diversity and severity of geologic conditions in Colorado that are development constraints.

Fields of Practice

Interview and survey responses revealed several areas of specialization that are emerging or becoming more important to engineering geology:

- Site characterization, especially using new remote sensing and imaging techniques. High resolution digital elevation models (DEMs) can also allow for general site characterization studies, but cannot be allowed to replace a field visit.

- Feasibility studies, hazard identification and mitigation design studies for residential development, infrastructure and civil projects (highways, dams, power-generating and transmission facilities), with an emphasis on economical mitigation methods, and better communication between geologists, engineers, developers and owners.

- Geological hazard evaluation that focuses on risk assessment, using more quantitative and predictive analyses, e.g., evaluating the probability of debris flows, and calculating return intervals, similar to flood hazard and seismic hazard analyses.

- Development of excavation and ground support methods to enable safe and reliable earthwork and construction on difficult sites.

- Prediction and mitigation of ground failures, especially those caused by collapsible, corrosive or low-strength soils possessing soluble minerals or other adverse geochemistry.

- Evaluation of the water quality impacts related to oil shale and coalbed methane resource development.

Bar charts summarizing survey respondents' opinions on emerging fields of practice over the next 5 and 20 years are presented on Figure 1. The data indicate that Colorado's water resource and infrastructure needs are expected to keep many of us busy for the foreseeable future. To a

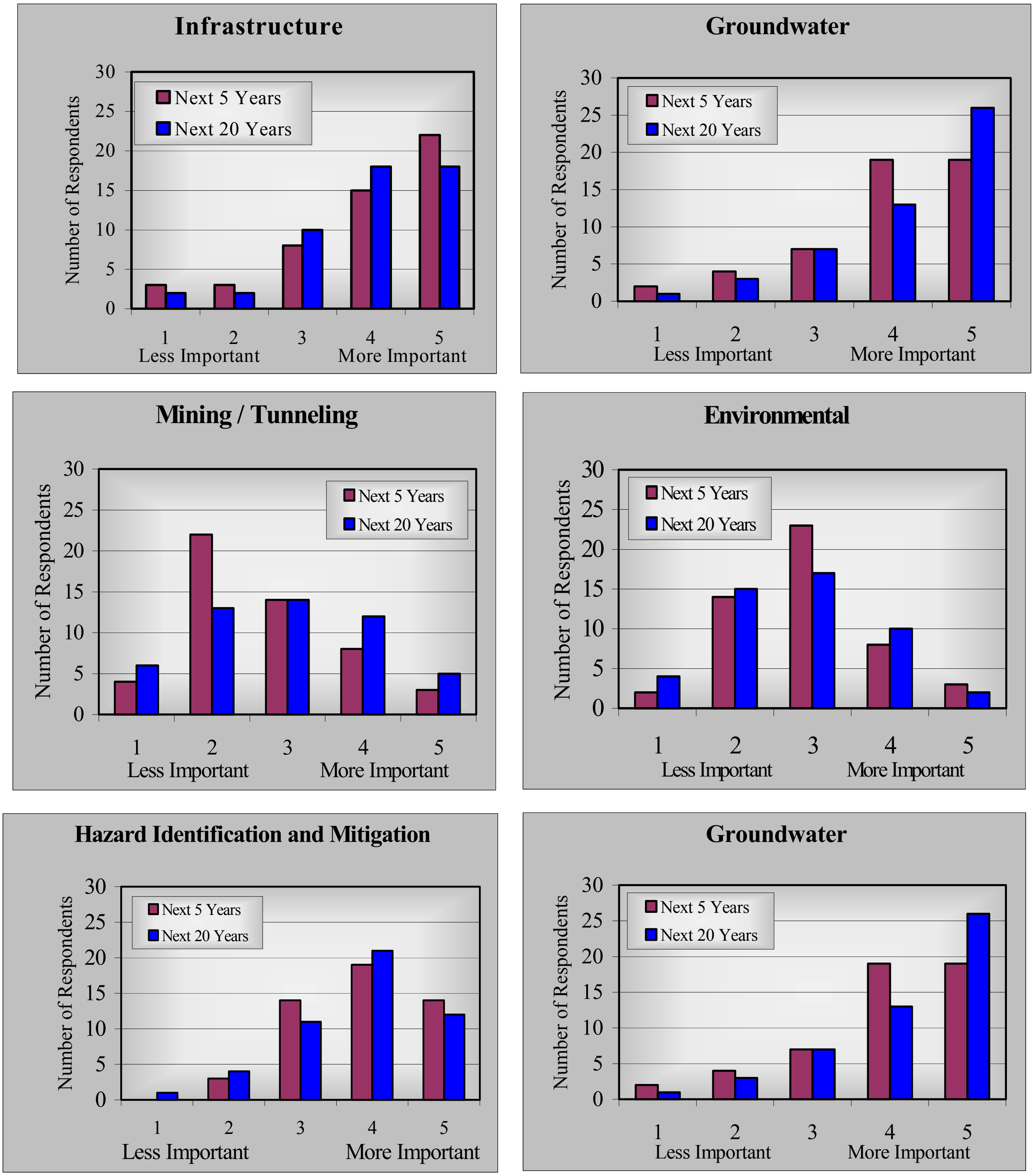

Figure 1. Importance of Fields of Practice

lesser extent, geological hazard identification and mitigation will continue to be increasingly important areas of practice.

Seismic issues are coming to the frontline at both the state and regional level. As data is collected regarding seismic hazards in Colorado, we are learning that Colorado is at greater seismic risk than originally believed, and many engineering geologists are not aware of this. Most of this research is being and will continue to be performed by government agencies and academia, and the question of funding is becoming increasingly important. Earthquake risk maps are available. However, Colorado needs to incorporate more accurate data into its seismic hazard maps to accurately represent the level of seismic risk that may be present in this state. Until the International Building Code is revised to incorporate site-specific earthquake studies in Colorado, the private sector will likely not evaluate this hazard on a site-specific basis.

Liquefaction is a hazard that has received only limited consideration in Colorado. Areas on the western slope are underlain by soils that have been saturated by irrigation and may be susceptible to earthquake-induced liquefaction.

Rockfall is becoming a bigger issue as highways are widened to accommodate the increase in high country traffic. Rockfall accidents and fatalities, such as the 1999 incidents on the Georgetown Incline, have sparked an effort to evaluate rockfall hazard areas. At present, limited funding is available to remediate the high-risk rockfall areas. Similarly, the identification and mitigation of debris flow and avalanche hazard areas will become more important as development spreads into the mountains.

The hydrogeology sector of the practice feels that the field of water resources is becoming more important, and non-traditional methods of water collection, such as rainwater harvesting and use of coalbed methane discharge water should be considered. Water rights are an important related topic, and should be better understood by our profession. Identifying sites for surface water storage is becoming more important, and it is likely that the feasibility of subsurface water injection and storage will eventually be considered. Since many petroleum geologists have experience with fluid flow it will be interesting to see whether engineering geologists or petroleum geologists will be future players in this field.

Education and Training

Most respondents agree that engineering geology students should pursue a multidisciplinary course of study, including more courses such as engineering principles, soil mechanics, foundation engineering, and materials science. Engineering geologists who are familiar with properties of engineered materials (e.g. steel and concrete) are valuable assets during the design process. A working knowledge of GIS is becoming critical.

The trend toward omitting engineering geology as a required course in civil engineering programs (Rogers, 2002) virtually ensures that the engineering industry's perception of the importance of thorough site characterization and hazard identification will continue to decline. Some schools focus on an engineering curriculum at the undergraduate level and skip core geology courses. Conversely, many general geology programs seem to have adopted the philosophy that they exist to educate, but not necessarily to serve an industry. That is, some programs teach "geology trivia" rather than practical and applied concepts that are important to the field of engineering. Schools should encourage their civil engineering students with an interest in geotechnical engineering to pursue more core geology courses.

Many feel that we need at least a master's degree to do design work. A bachelor's degree usually confines the holder to the "technician" level. Those pursuing advanced degrees, and their advisers, should recognize that outside of academia, breadth of knowledge is often more important than depth of knowledge, and some professionals feel that it would be more useful to have two masters degrees, in engineering and engineering geology, than to have a PhD.

Employers continue to stress the need for improvements in technical writing skills, and in trench and borehole logging abilities. Students and newer employees need more training in how to create accurate and descriptive geologic logs, and in presenting their findings.

Considering the critical importance of field training and experience in mapping, drilling, sampling and geophysical methods related to environmental and engineering geology, many professionals would like to see longer field programs. Unfortunately, some schools are reducing departmental requirements and the amount of time devoted to field camp. One possible solution would be to offer two field camps: a classical geology field camp at the undergraduate level, and specialized engineering or environmental field camps or field courses, related to a student's specific area of study, at the graduate level. Another solution has been developed at the Colorado School of Mines, where students have specific portions of their field camp dedicated to engineering projects.

Colorado needs to strengthen its K-12 geology programs. CSAP-oriented curricula and teaching methods are eroding the level of science being taught and will eventually cause a decline in proficiency within our profession.

A coordinated mentoring program, possibly through professional associations and universities, would benefit geologists new to Colorado or the profession.

Suggested future short course or Geohazard conference topics include:

- Case histories. These provide a valuable perspective on how the field of engineering geology has developed.
- Geologic hazard identification and mitigation methods.
- Technology transfer sessions on ground improvement methods such as compaction grouting, soil nailing and excavation support, and new construction technologies such as MSE (mechanically stabilized earth) walls.
- For planners, an explanation of why and how a city develops in response to geological constraints, with case histories of successful and less successful examples.

Technology

Advances in geophysical methods, data collection, computer speed and memory have changed the way engineering geology is performed and presented in recent years. Bar charts summarizing survey respondents' opinions on areas of technical advance over the next 5 and 20 years are presented on Figure 2. The survey results indicate that no single area of technological advancement is expected to revolutionize the practice of engineering geology, but that we expect improvements in testing methods and data collection, processing and presentation to continue.

Many interviewees and survey respondents mentioned the increasing importance of GIS. "Smart," or georeferenced (e.g. ArcInfo) data will continue to be much more useful than data produced with computerized drafting programs such as AutoCAD.

Data collection methods are expanding to include:

- Satellite imagery. The use of satellite-based sensors could help to provide regional subsidence or heave rates that can be used as background data for comparative studies.
- Electronic field equipment such as ArcPad computers and GPS receivers for mapping (See adjacent photo).
- Real-time and remote monitoring, providing a vast amount of data that previously was difficult to collect and interpret.

ArcPad handheld computer and GPS receiver in use. Arial photos can be loaded into ArcPad and used as a base map.

Hand-held spectrometers and hyperspectral technology could help with on-site clay analyses in a relatively fast and cheap manner. Although engineering properties of clays and claystones have proven to be important development factors along the Front Range and other areas underlain by shales, many geotechnical engineers do not perform enough testing to adequately evaluate the risks of structural damage due to expansive and collapsible soils.

Site characterization abilities will improve as technology becomes more sophisticated, but engineering geologists cannot let flashy technology take the place of a thorough field exploration program. We also cannot let ourselves be mesmerized by fancy maps – they may be a product of very limited basic data whose inaccuracy or inapplicability may be obscured by the slick presentation.

Specialized equipment is becoming available that allows access to, and even remote operation within, steep and hazardous areas for drilling, sampling and earthwork.

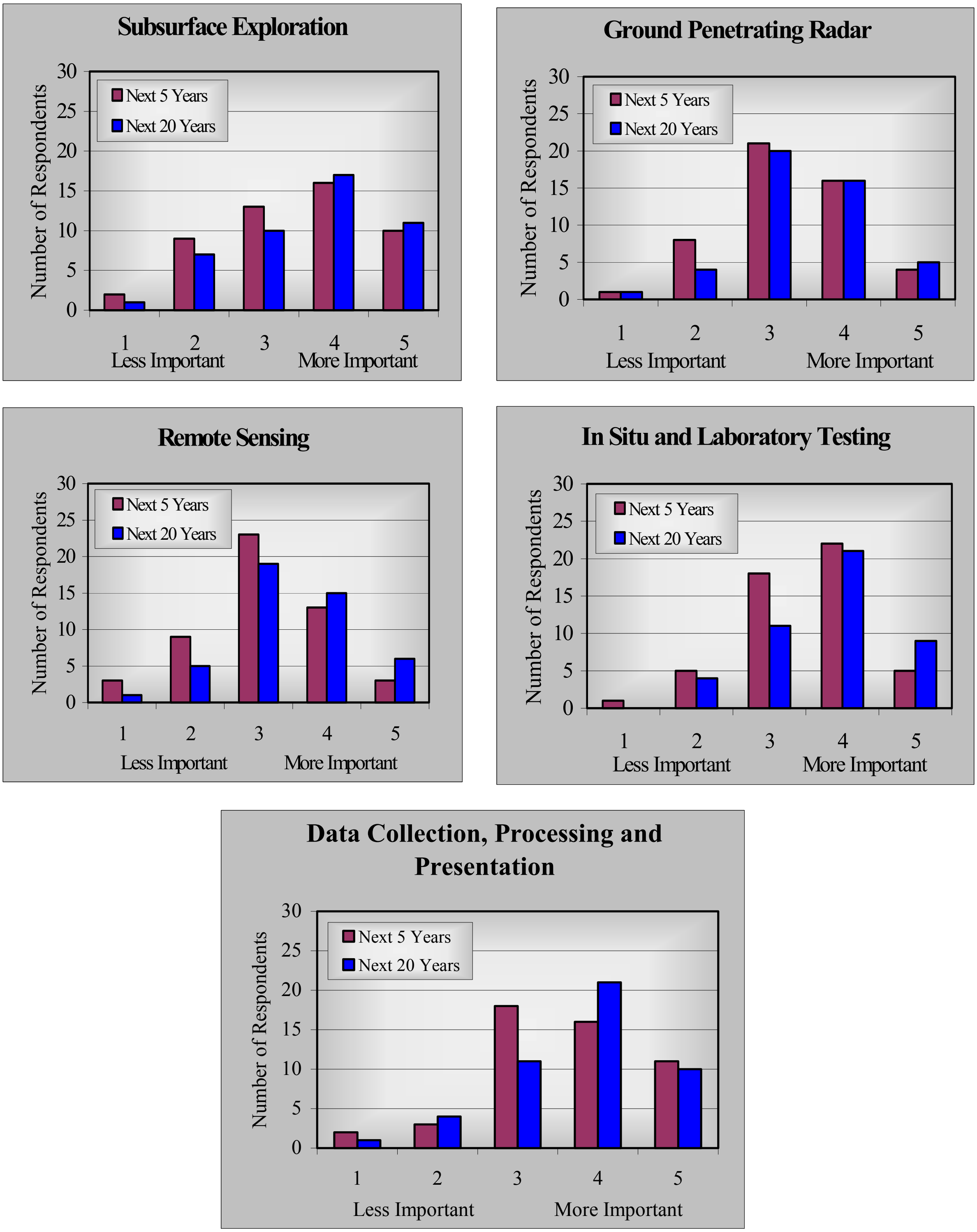

Figure 2. Importance of Technical Advancements

Soil nail walls, tieback systems and ground anchor walls for landslide mitigation are examples of new site remediation methods that are being used in Colorado. MSE walls are replacing traditional cantilever systems. One very new method being used in Colorado for ground stabilization is micropiles as temporary excavation support in place of caissons. Micropiles allow the earth to "support itself" during construction.

Geophysics is being used more in tunneling and subsurface work in urban areas to locate obstructions and weak rock areas. However, there is room for improvement of "old" technologies such as ground penetrating radar and other geophysical methods.

Resources

The quality and quantity of available resources related to the engineering geology of Colorado is variable, but improving. Although a vast amount of information is available at the USGS library, many more reports are private property and not available for public use or very difficult to find. All interviewees and survey respondents agree that it would be useful to have an online, statewide database of available maps and publications maintained by an agency such as CGS or the Colorado Department of Transportation (CDOT).

Thesis documents are good sources of site-specific information. Most of these resources are completed as a response to an event and are somewhat limited in scope, but nonetheless may provide useful background information.

The CGS has also received several requests for a reissue of Trimble and Machette's 1979 Front Range Corridor map. It would be useful to repackage some of the existing data (such as county 1041 geologic hazard and resource maps and older, out of print geologic maps) into a GIS or other digital format that would be easier to distribute, more user-friendly, and would facilitate revisions and updates.

Communication

Communication between and among engineers, geologists, developers, contractors, government agencies, and the public has been, at times, erratic. Timely and widespread dissemination of essential information is also a tricky subject, as consultants who have certain data may be reluctant to share with competitors, and government agencies can be slow and underfunded.

Concern has been expressed regarding the collection and publication of accurate geologic/engineering geology mapping and studies that are unbiased by political agendas. We need to strengthen the professional engineering community's opinion regarding the value of geologic interpretation – too many engineers view geology as a set of static conditions, without considering the many dynamic geomorphological processes that can affect their projects.

Networking is a valuable tool for exchanging knowledge and continuing to improve the standard of practice. AEG, Colorado Association of Geotechnical Engineers (CAGE) and American Society of Civil Engineers (ASCE) meetings are valuable resources that more engineering geology professionals should avail themselves of.

Also needed is clarification of what areas should be declared or remain non-buildable. There is heavy development pressure in areas that were once deemed non-buildable, and consultants are sometimes reluctant to state that an area is indeed undevelopable. While it is true that new technologies may allow for mitigation of problems that at one time may have rendered a project infeasible, regional impacts should be considered also. Others feel that there is no such thing as a non-buildable site. Clearly, there is room for cooperation between engineering geologists (experts in hazard identification), and geological engineers (experts in hazard mitigation).

Public disclosure is also becoming an important topic. As more and more "difficult" terrain is being developed, public exposure to geological hazards is increasing. Homebuilders are faced with lawsuits concerning existing and potential geologic hazards that were not disclosed. Some claims are unreasonable and are not based on science but politics and emotions.

CONCLUSION

The future of engineering geology in Colorado promises to be interesting. There remains much room for improvement in the areas of site characterization, hazard identification and mitigation. Wise and informed land use and resource development, and advances in investigation and data collection methods can provide us with the potential ability to work more accurately, efficiently and effectively.

In order to ensure a healthy future for the profession, engineering geologists should strive to be proactive, identifying and helping to develop solutions for geologic constraints that affect existing and planned development in Colorado. We should work to educate the public and our colleagues in education, industry and government about the value of our expertise in helping to provide safe and cost-effective solutions to geological challenges facing Colorado.

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APPENDIX I: ONE-PAGE SURVEY

Dear Colleague:

The Engineering Geology Section of the Colorado Geological Survey is conducting a survey and interviews on the **Future of Engineering Geology Practice in Colorado** for an upcoming AEG (Association of Engineering Geologists – Rocky Mountain Section) publication. We are interested in your thoughts! Please take a few minutes to answer some short questions for us. If you have comments or questions, please feel free to contact Jill Carlson, Sean Gaffney or TC Wait at (303) 866-2611 or at http://geosurvey.state.co.us/.

> **Please return to the publications table** at the workshop before you leave, or by October 31, 2002 via…
>
> Fax: 303.866.2461
>
> Or mail: Colorado Geological Survey
> 1313 Sherman Street, Room 715
> Denver, CO 80203
>
> **Thank you!**

1. Please rate the following fields of practice by their importance to the future of engineering geology in Colorado…
(**1** indicates *lowest* level of importance and **5** is the *highest*.)

	over the next 5 years	over the next 5 to 20 years
Infrastructure development	1 2 3 4 5	1 2 3 4 5
Groundwater resource development	1 2 3 4 5	1 2 3 4 5
Mining and tunneling	1 2 3 4 5	1 2 3 4 5
Environmental testing & remediation	1 2 3 4 5	1 2 3 4 5
Hazard identification & mitigation design	1 2 3 4 5	1 2 3 4 5
GIS	1 2 3 4 5	1 2 3 4 5

2. Please rate the following areas of technical advancement by their importance to the future of geological engineering…

	over the next 5 years	over the next 5 to 20 years
Subsurface exploration & sampling	1 2 3 4 5	1 2 3 4 5
Geophysics	1 2 3 4 5	1 2 3 4 5
Remote sensing	1 2 3 4 5	1 2 3 4 5
In situ and lab testing	1 2 3 4 5	1 2 3 4 5
Data collection, processing and presentation	1 2 3 4 5	1 2 3 4 5

3. Please help us focus our interviews by sharing your thoughts on the future of engineering geology practice in Colorado. Possible topics include: future **trends** in engineering geology, **technologies** that are becoming available, ways we can better **communicate** with and benefit each other and the public, areas that you feel need the most **strengthening** in the future, **geographical** areas of increasing or decreasing importance, trends in **education**, **licensure**, or **anything else** you have an opinion on, provided it's at least tangentially related to the future of engineering geology. *You don't have to limit yourself to this tiny space* – feel free to write as much or as little as you want, and fax or e-mail your reply to jill.carlson@state.co.us

____

____

____

4. Which of the following best describes your occupation?

___ Geologist ___ Engineering Geologist ___ Engineer ___ Other, please specify ________________

5. In which region of Colorado do you do the majority of your work?

___ Front Range ___ Denver Metro ___ Colorado Springs / Pueblo ___Grand Junction & vicinity
___ Central Mountains ___ North Central Mountains ___ San Juan Mountains & Southwest
___ Southeast ___ Northeast ___ Other ________________

6. How long have you practiced engineering geology or a related field…

…in Colorado? ___ 0 - 5 years ___ 5 – 10 years ___ 10 – 20 years ___ 20 or more years
…in any state? ___ 0 - 5 years ___ 5 – 10 years ___ 10 – 20 years ___ 20 or more years

7. How would you describe your receptiveness to being interviewed by CGS regarding your views on the future of engineering geology?

___ Sure, I have some good thoughts on the future of engineering geology in Colorado and would be pleased to share them with one of the fine folks at CGS.
___ OK, but it better be quick, I'm very busy. And only if CGS is buying lunch.
___ Not a chance.

____

Name Phone E-mail Time best reached…

ENGINEERING GEOLOGY AND HYDROGEOLOGY EDUCATION IN COLORADO

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Department of Geology and Geological Engineering, Colorado School of Mines, Golden, CO 80401

***Key Terms*: education, universities, colleges, continuing education**

ABSTRACT

Colorado has a critical combination of elements that call for talented engineering and ground-water geologists: rapid growth, significant geologic hazards, and mounting environmental and water supply concerns. In Colorado, professional geologists and geological engineers have regular occasions to expand their skills and learn state-of-the-art techniques, and college students have several options available to prepare them for careers in engineering and ground-water geology. Four universities teach courses in engineering geology, eleven teach classes in ground water, three offer environmental sciences or environmental geology specialty tracks, and one offers a geological engineering degree specific to engineering geology and hydrogeology. The state geological survey provides regular publications and hosts meetings covering engineering geology topics, and two important professional organizations (GSA and AEG) have a strong presence in the state. Several geology organizations maintain their headquarters in Colorado, and several federal agencies have important regional offices, research groups, and resource centers in the state.

INTRODUCTION

As a state with rapid growth, significant geologic hazards, and mounting environmental and water supply concerns, Colorado has a strong need for vigorous education in engineering and ground-water geology. Professional geologists and geological engineers have ample opportunities to expand their skills and learn state-of-the-art techniques, and college students have several options available to prepare them for careers in engineering geology and ground water. Many of these opportunities are described below, and implicit in these descriptions is the backdrop of the state as a natural laboratory, where rock units and geologic processes may be easily viewed at the surface, and within close proximity to major cities.

UNIVERSITY CLASSES AND FACULTY IN ENGINEERING GEOLOGY

According to the university web-sites and catalogs, four universities or colleges in Colorado offer a specific class in engineering geology and 11 offer a class in ground water (shown on Table 1). Twelve schools offer other courses related to engineering geology and ground water including, environmental geology, geomorphology, and soil and rock mechanics. The courses are generally taught in Geology or Geological Engineering programs, but may also be taught through Civil or Mining Engineering or Geography programs.

The research and teaching interests of faculty are also summarized in Table 1. There is not always a match between course offerings and faculty expertise, as many of these courses are taught by individuals who do not list the topics as their primary expertise (as recorded in AGI, 2002) or even as an "area of interest" (as recorded in the university's web page or catalog).

Table 1. Summary of Engineering Geology Offerings at Colorado Colleges and Universities.

College or University	Courses related to engineering geology						Number of full-time faculty listing subject as primary expertise (numbers who listed subjects as "interests" are in parentheses)			
	engineering geology	environmental geology	geomorphology	ground water	soil mechanics	rock mechanics	engineering geology	geomorphology	ground water	environmental science / environmental geology
Adams State College (Alamosa)			●	●						
Colorado College (Colorado Springs)		●	●							
Colorado School of Mines (Golden)	●		●	●	●[1]	●[2]	3	(2)	2	
Colorado State University (Fort Collins)		●	●	●	●[1]			2	1	
Fort Lewis College (Durango)	●		●	●			(1)	1	(1)	(1)
Mesa State College (Grand Junction)		●	●	●			(1)	1	(1)	(1)
Metropolitan State College of Denver			●	●						
University of Colorado at Boulder	●[1]	●	●	●	●[1]	●[1]		1	1	
University of Colorado at Denver	●	●	●	●	●[1]	●[1]				1
University of Denver		●[3]	●[3]	●[3]				2		1
University of Northern Colorado (Greeley)			●	●				1		
Western State College (Gunnison)			●	●				1		

[1]taught through Civil Engineering department
[2]taught through Mining Engineering department
[3]taught through Geography department
Data collected from college and university web sites, catalogs, and AGI (2002)

ENGINEERING GEOLOGY DEGREE PROGRAMS

Of the 12 schools listed in Table 1, three offer environmental sciences or environmental geology specialty tracks for undergraduates, and one offers a geological engineering degree specific to engineering geology and hydrogeology. Three of the four schools offer graduate degrees in engineering geology, hydrogeology, or related fields. Each program is summarized below.

Colorado School of Mines

The Colorado School of Mines in Golden is the only university in the state that offers a full degree program preparing students for careers in engineering geology. For this reason, this program is discussed in more detail than the other programs in Colorado. The department of Geology and Geological Engineering offers an ABET-accredited undergraduate degree in Geological Engineering, with emphasis in engineering geology and ground water or emphasis in petroleum and minerals exploration. Graduate degrees (M.S. and Ph.D.) are offered in Geology and in Geological Engineering, with the latter oriented towards engineering geology or hydrogeology practice. The undergraduate degree provides the background for students to work towards registration as both geologists and engineers, and the engineering geology and ground water track consists of the following general elements:

math, programming, and basic sciences	37 hours
humanities, social sciences, economics, and electives	30 hours
basic engineering and engineering practices	18 hours
geology	41 hours
engineering geology, hydrogeology, and geomechanics	21 hours
Total	147 hours

Students are required to take courses in engineering geology and geotechnics and in ground-water engineering. Other required courses related to those fields include geomorphology, engineering geology design, ground-water engineering design, soil mechanics, rock mechanics, fluid mechanics, statics, mechanics of materials, and thermodynamics. The two design courses are taken during the final semester and require the students to apply their engineering and geologic skills to solve real-world engineering geology and hydrogeology problems (Figure 1).

Figure 1. Students completing a debris fan logging project.

At the graduate level, students complete a degree in Geological Engineering, which includes the background courses to prepare them for the Fundamentals of Engineering and Fundamentals of Geology Exams. Most students complete a research thesis as part of their degree, although a non-thesis option has been recently developed. A broad range of courses is available for students in these programs. Many of the undergraduate courses listed previously are suitable for graduate students. At the graduate level, students may select courses in:

site investigation
risk assessment
advanced hydrogeology
geographic information systems
geological data analysis
case histories in geological engineering and hydrogeology
advanced engineering geology
landslides
advanced geotechnics
advanced ground-water engineering
ground-water modeling
vadose zone hydrology

Other courses of interest are offered through departments of Civil Engineering, Mining Engineering, and Environmental Science and Engineering. These departments also house faculty members with expertise in ground water, contaminant hydrogeology and soil and rock engineering.

The department of Geology and Geological Engineering maintains a vigorous research program in engineering geology and ground water, with recent research grants and student support from the National Science Foundation, Transportation Research Board, Bureau of Indian Affairs, U.S. Geological Survey, Colorado Division of Water Resources, U.S. Department of Defense, U.S. Department of Education, and U.S. Environmental Protection Agency, among others. The International Ground-Water Modeling Center, which offers numerous short courses and distributes a wide variety of ground-water software, is headquartered in the department.

For engineering geology and hydrogeology, the department has awarded an annual average of 14 BS degrees, 8 MS degrees, and 2 Ph.D. degrees since 1990.

Colorado State University

The Colorado State University, located in Fort Collins, offers an Environmental Geology concentration for undergraduates in the Geosciences department. In addition to traditional geology and science courses, students following this track take classes in environmental geology, hydrogeology, geomorphology, soil science, biology, and earth resources. The degree requires 120 hours of coursework.

The graduate program in the department offers specializations in Environmental Geology, Hydrogeology, and Geomorphology. The department declares a “strong emphasis on field-oriented studies in non-renewable resources and surficial processes (CSU, 2003a)." Supporting faculty and courses are available in the departments of Civil Engineering (which includes areas of study in environmental engineering, ground-water hydrology, water resources, and geotechnical engineering) and Soil and Crop Sciences. The university houses the Colorado Water Resources Research Institute, whose mission is "focusing the water expertise of higher education on the evolving water concerns and problems being faced by Colorado citizens (CSU, 2003b)," through sponsored research.

Fort Lewis College

Fort Lewis College, in Durango, offers an Environmental Geology option for undergraduates in the Geosciences department. Courses related to engineering geology include geomorphology, engineering geology, and ground-water geology. The degree requires 128 hours of coursework.

University of Colorado at Boulder

The University of Colorado at Boulder offers an Environmental Sciences option for undergraduates in the Geological Sciences department. The students are required to take hydrogeology, and then may choose electives to focus on "hydrology aspects," "chemical aspects," "physical geology aspects," or "global Earth systems aspects." The degree requires between 119 and 123 hours of coursework.

The graduate program in the department offers a specialization in Hydrogeosciences. The goal of this program is to "understand the processes of surface-water hydrology, hydrogeology, including groundwater quality, groundwater-surface water interaction, fracture flow, and perched water formation (CU, 2003)." Supporting faculty and courses are available in the department of Civil, Environmental, and Architectural Engineering.

CONTINUING EDUCATION OPPORTUNITIES

There are several avenues of continuing education for engineering geologists in Colorado, in the forms of meetings, field trips, and publications. A well-attended event is the annual meeting of the Geological Society of America (GSA), which is held in Denver every two to three years, rotating to other cities in off-years (recent Denver meetings were 1999 and 2002, and scheduled future meetings are in 2004 and 2007). This meeting generally features several technical sessions and field trips sponsored by the Engineering Geology Division of GSA.

The Rocky Mountain Section of the Association of Engineering Geologists encompasses Colorado, as well as New Mexico and parts of Wyoming, Montana, and South Dakota. The section is very active, publishing a newsletter and meeting monthly, except during the summer. The section is hosting the annual meeting of the AEG at Vail in 2003. The Colorado School of Mines hosts a student chapter of AEG that meets approximately 15 times each year (Figure 2).

Figure 2. Participants in the Rocky Mountain Section Student Night 2003.

The Colorado Geological Survey (CGS) publishes a quarterly newsletter, “RockTalk,” that often features engineering geology and hydrogeology topics, such as geologic hazards (April 1998), engineering geology and geologic hazards (October 1999), ground subsidence (October 2001), Colorado earthquakes (April 2002), and ground water (October 2002). These newsletter issues may be downloaded from the CGS website (http://geosurvey.state.co.us/pubs/rocktalk/ rocktalk.htm). The CGS also publishes maps, books, and field trip guides covering a wide range of engineering geology topics in the state (a catalog may be viewed at their website http://geosurvey.state.co.us/). Since 1996, the CGS has hosted annual conferences covering a variety of geologic and hydrologic issues, highlighting those most prevalent near the host cities (which have included Colorado Springs, Glenwood Springs, Montrose, Durango, and El Jebel near Carbondale).

The Colorado Association of Geotechnical Engineers (CAGE), in conjunction with the Rocky Mountain AEG Section and the Geotechnical Section of the American Society of Civil Engineers (ASCE), offers a one-day, biennial meeting on geotechnical topics, covering such topics as “Applying Geotechnical Engineering to Construction” (1996), “GeoData” (1998), and “Geotechnical Engineering in Transportation” (2002).

AGENCIES / ORGANIZATIONS IN COLORADO

There are a multitude of geology and engineering consulting firms with headquarters or offices in Colorado. Also, there are a number of governmental agencies that conduct engineering geology work with offices in the state, and several national organizations serving engineering geologists who have their headquarters in Colorado. For example, the U.S. Geological Survey employs 1892 people in Colorado, where the organization maintains the Central Region Headquarters in the Federal Center (Lakewood), the Central Region Water Resources Division (Lakewood), the Central Region Geologic Hazards Team and Landslides Group (Golden, on the Colorado School of Mines campus), the National Earthquake Information Center and World Data Center for Seismicity (Golden, on the Colorado School of Mines campus), and a publication sales office (Lakewood).

The U.S. Bureau of Reclamation, which oversees the operation of many dams in the Central and Western U.S., maintains a headquarters office in Lakewood, Colorado, that employs a large engineering geology and geological engineering staff. The Colorado Geological Survey, headquartered in Denver, has both Engineering Geology and Ground-Water Sections. The Colorado Department of Transportation has a Geotechnical Section that conducts a substantial amount of engineering geology work. The U.S. Environmental Protection Agency (EPA) maintains the Region VI headquarters office in Denver.

The following organizations serving engineering geologists maintain their headquarters in Colorado:

Association of Engineering Geologists (Denver)
American Institute of Professional Geologists (Westminster)
Clay Minerals Society (Aurora)
Environmental and Engineering Geophysical Society (Denver)
Geological Society of America (Boulder)
Society for Mining, Metallurgy, and Exploration (Littleton)
Society of Economic Geologists (Littleton)

SUMMARY

Opportunities for engineering geology and hydrogeology education are abundant in a state where many geologic processes are readily visible. Four universities teach courses in engineering geology, three offer environmental sciences or environmental geology specialty tracks, and one offers a geological engineering degree specific to engineering geology and hydrogeology. The state geological survey provides regular publications and hosts meetings covering engineering geology topics, and two important professional organizations (GSA and AEG) have a strong presence in the state. Several geology organizations maintain their headquarters in Colorado, and several federal agencies have important offices, research groups, and resource centers in the state.

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EARTHQUAKE-INDUCED HAZARDS ALONG TRANSPORTATION ROUTES IN COLORADO

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Key Terms: earthquake, landslide, liquefaction, highways

ABSTRACT

Recent research by the Colorado Geological Survey and others has raised the level of awareness of earthquake potential in Colorado. However, little work has been done to evaluate the vulnerability of transportation routes to earthquake hazards. A preliminary study has been completed to assess the potential for earthquake related damage to roadways, and to identify critical areas for more detailed study. The hazards evaluated include liquefaction and landslides. Both processes will be most pronounced in the western mountains and plateaus, where anticipated earthquake accelerations are the highest.

Liquefaction can be expected for typical sandy saturated soils with corrected SPT values in the range of 5 to 10 exposed to smaller earthquakes, or SPT values of 5 to 23 exposed to larger earthquakes. These soils would include young sands, silts and gravels in river valleys and basin fill alluvium of the San Luis Valley. River valleys that may be exposed to the required level of earthquake shaking include the Eagle, Colorado, Gunnison, Uncompahgre, Animas, Los Pinos, and Upper Rio Grande. Potentially vulnerable highways include Interstate 70; U.S. Highways 24, 50, 160, 285, 550; and numerous State Highways. Potentially affected communities include Delta, Montrose, Ridgway, Durango, Gunnison, Creede, Del Norte, and the San Luis Valley (particularly the northern and southern ends).

A series of design charts and graphs were developed to assess landslide hazards, which may be evaluated in terms of slope angle, earthquake acceleration, and soil strength. In general, small landslides (less than 100 cubic yds or 75 cubic m) are expected to occur in steep slopes (30 to 40°), with weak, soils (cohesion less than 100 to 200 psf (5 to 10 kPa), friction angle less than approximately 20°), with earthquake accelerations on the order of 10%g. Large landslides (greater than 10,000 cubic yds or 7500 cubic m) are expected to occur in shallower slopes, with stronger soils, and at lower earthquake accelerations than for small slopes. The most vulnerable slope materials are expected to be saturated clayey soils, such as residual soils developed over fine-grained sedimentary bedrock, and thick alluvial or glacial sediments with low to moderate strength.

INTRODUCTION

Transportation routes are a particularly vulnerable link in our ability to respond to and recover from earthquakes. They are sensitive because they must cross all types of terrain and geology, yet they are generally not considered critical enough to warrant expensive earthquake-resistant construction techniques. They are essential for access in and out of stricken areas for movement of emergency supplies and manpower, and are vital to sustain the areas during recovery.

The linear nature of transportation routes means that a single route can be exposed to a wide variety of hazards, and that a problem anywhere along the route will preclude its use for miles in either direction. Landslides induced by earthquakes can bury or undermine highway lanes and rail lines, tilt bridge piers, or block streams and flood adjacent transportation routes (Beavers, 1981; Hopper, 1985). Liquefaction of sandy or silty soils can result in settlement and warping of roadways, cracking and separation from lateral spreads, or flooding due to subsidence of low-lying areas (Hopper, 1985; Wheeler, et al., 1984). Failure of levees and dams due to seismic shaking can flood transportation routes and receding waters can deposit thick sheets of sediment on these routes (Hopper, 1985; Santi and Neuner, 2002). Strong ground motion alone can damage bridges, induce rockfall onto routes, crack roads, and uplift and warp roadways and rail lines (Beavers, 1981; Wheeler, et al., 1984). Similarly, surface fault rupture can render routes impassable until they are realigned.

The purpose of this study is to quantify the potential effects of earthquakes on transportation routes in Colorado, and to identify the most vulnerable areas for further study. The study will be restricted to liquefaction and landslide susceptibility.

BACKGROUND

Recent research by the Colorado Geological Survey (CGS) and others has raised the level of awareness of earthquake potential in Colorado. For example, the CGS has recently produced a compendium of Colorado earthquake information (Kirkham and Rogers, 2000), a map of Quaternary faults in the state (Widmann, et al., 1999), and an issue of their quarterly newsletter dedicated to earthquakes (CGS, 2002). These documents confirm a strong potential for damaging earthquakes in the state: over 92 faults with Quaternary movement have been catalogued, historic earthquakes with magnitudes as high as 6.6 have been recorded, and maximum credible earthquakes as large as M = 7.5 have been assigned to Colorado faults (CGS, 2002).

In spite of its importance, research quantifying the vulnerability of transportation routes to earthquake damage is rare. Santi and Neuner (2002) evaluated hazards for a highway in Missouri that was declared to be an emergency rescue route and therefore of high priority to keep operational following an earthquake. Quantitative studies have been completed for transportation routes in the New Madrid Seismic Zone (FEMA, 1990; Hopper, 1985; Wheeler, et al., 1994), the Eastern U.S. (Beavers, 1981), and California (O'Rourke, et al., 1991, for example). Qualitative studies have been completed for infrastructure for the United States as a whole (FEMA, 1987; FEMA, 1991; Hays, 1987; Scawthorn, et al., 1992). None of these studies have directly addressed Colorado, and they provide at most expected earthquake shaking levels and a qualitative assessment of geologic hazards. This study will provide a quantitative evaluation of liquefaction and landslide hazards to transportation routes, concentrating on soil behavior.

LIQUEFACTION SUCEPTIBILITY

Liquefaction is the settlement and accompanying water movement when earthquake shaking densifies saturated non-cohesive materials. It results in cracking of structures due to differential settlement and lateral spreading of shallow soils, ejection of large sand boils onto the ground surface, and potential subsidence and flooding. An analysis is presented below to estimate the critical Standard Penetration Test (SPT) values of soils expected to liquefy under various earthquake scenarios.

Methods

Liquefaction susceptibility was calculated using the method developed by Seed and Idriss (Idriss, 1999). As a basis to represent a "typical" soil profile, a hypothetical unit was analyzed at a depth of 20 ft (6m) below ground surface, with a water table at 10 ft (3m) below ground surface. The soil was assumed to have a moist density of 100 pounds per cubic ft (pcf) (1600 kg/m^3) and a wet density of 120 pcf (1900 kg/m^3).

Expected earthquake accelerations were obtained from USGS (2003). Accelerations were selected for earthquakes with 10 percent (Figure 1) and 2 percent (Figure 2) probabilities of exceedance (PE) in 50 years. The assumed earthquake magnitude, using information from Colorado Geological Survey (2002), is 7.0.

The critical SPT values were calculated from Seed and Idriss' equations (Idriss, 1999):

$$\text{FS} = \text{Corrected CRR} / \text{CSR} \qquad (1)$$

where:

FS = Factor of Safety (assumed equal to 1.3, as suggested by CDMG, 1997)

Corrected CRR = Corrected Cyclic Resistance Ratio (ability of the soil to resist shaking). CRR is based on the graphs in Seed and others (1985), which have been converted to equations 2, 3, and 4 below. The correction factor is for magnitude correction of strong ground motion, based on data presented in Seed and others (1985), converted to equation 5 below. The CRR and the correction factor are multiplied to produce the Corrected CRR.

CSR = Cylic Stress Ratio (produced by the design earthquake), calculated from equation 6 below.

CRR values are calculated as a function of fines content and $(N_1)60$ values (corrected SPT values) of the soil:

5% fines: $\text{CRR} = 0.0351e^{0.0914(N_1)60}$ (2)

15% fines $\text{CRR} = 0.0674e^{0.0787(N_1)60}$ (3)

35% fines $\text{CRR} = 0.078e^{0.0895(N_1)60}$ (4)

The magnitude correction factor is normalized to M = 7.5 and is valid for the range 4 < M < 9:

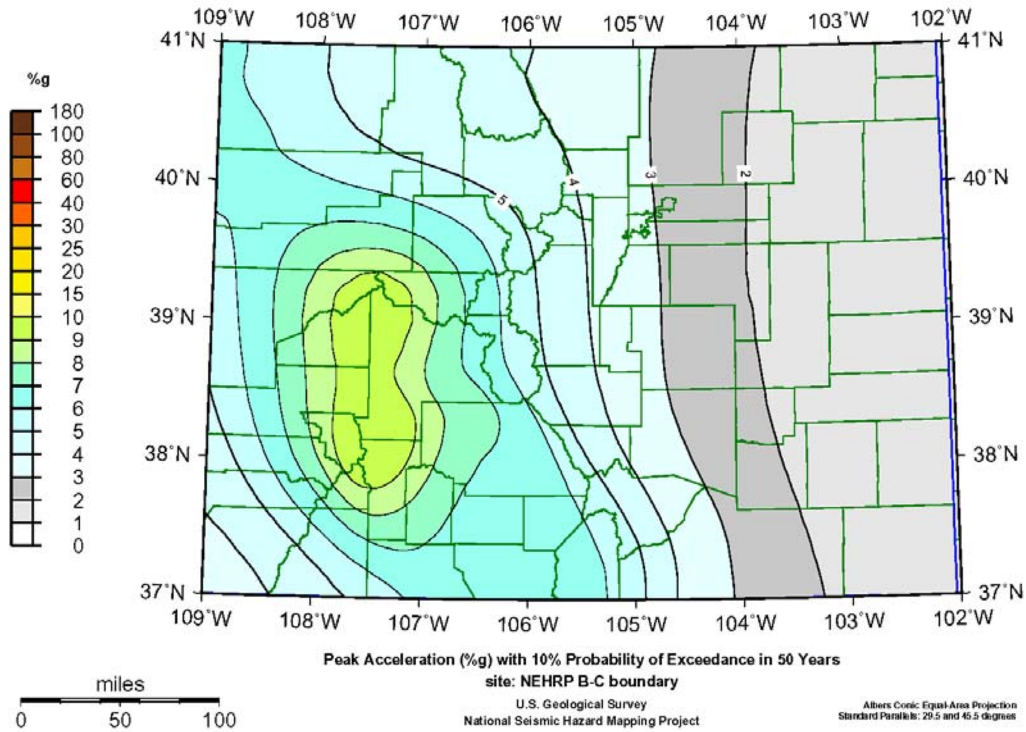

Figure 1. Expected earthquake accelerations with 10% Probability of Exceedance in 50 years (from USGS, 2003).

$$\text{Correction factor} = 9.5284M^{-1.1123} \qquad (5)$$

CSR may be calculated using the equation developed by Seed and Idriss (Idriss, 1999):

$$CSR = 0.65r_d(\sigma_0 / \sigma'_0)(a_{max}/g) \qquad (6)$$

where σ_0 is the total overburden stress, σ'_0 is the effective overburden stress, a_{max} is the maximum horizontal ground acceleration, and g is the acceleration of gravity. The value r_d is a depth correction factor calculated by the following equation:

$$r_d = 1 - 0.012z \qquad (7)$$

where z is the depth of the layer of interest, in meters.

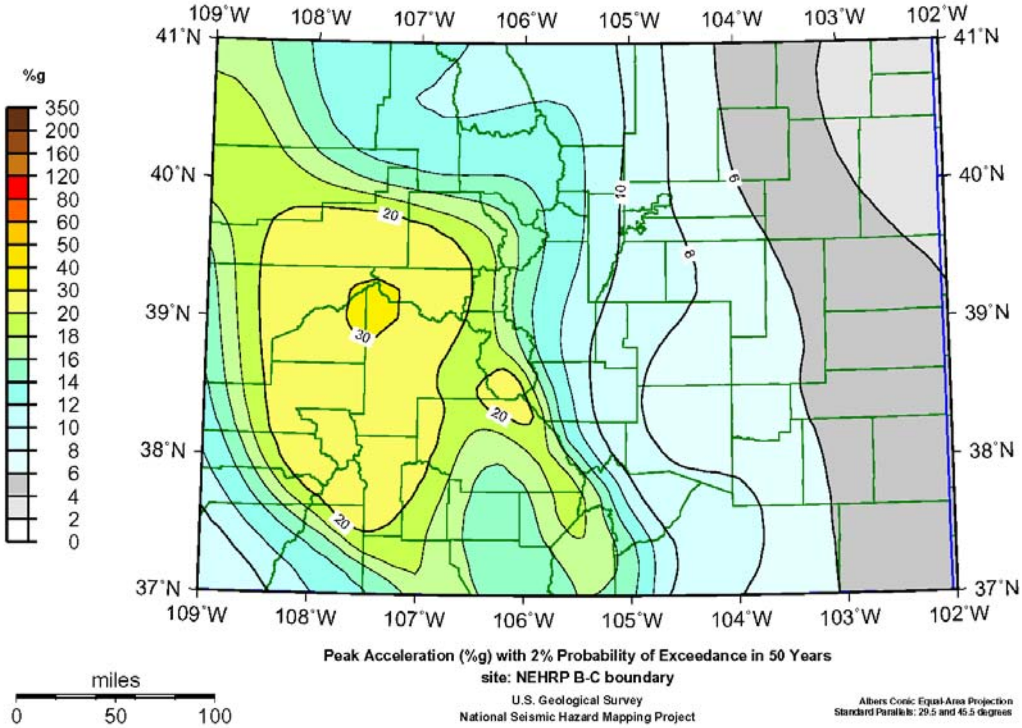

Figure 2. Expected earthquake accelerations with 2% Probability of Exceedance in 50 years (from USGS, 2003).

The equations above were combined, solved for $(N_1)60$, and entered into a spreadsheet to streamline calculations. Various values for earthquake acceleration were then entered into the equation and the resulting critical $(N_1)60$ values (below which liquefaction could be expected to occur) were recorded. It should be noted that the National Research Council (1985) suggests a minimum threshold acceleration of 10%g is necessary to induce liquefaction. However, for the sake of completeness, lower values of acceleration were also evaluated in this study.

Sensitivity Analysis

Because such a large area with widely varying conditions was evaluated in this study, a sensitivity analysis was completed to identify the importance and influence of each parameter included in the liquefaction analysis. The first goal of the sensitivity analysis was to identify parameters that had minor influence on the overall analysis so that the “typical” soil profile and earthquake conditions outlined above could be considered representative without changing these parameters. The second goal of the sensitivity analysis was to estimate the corresponding change in critical SPT values for an incremental change in each input parameter.

Using the "typical" soil profile and earthquake conditions as default values, one parameter at a time was varied across a reasonable range of expected values for that parameter. The parameters changed and their range of variation are as follows:

- factor of safety – varied from 0.8 to 1.8,
- earthquake acceleration – varied from 2 to 40%g,
- earthquake magnitude – varied from 4 to 9,
- depth of soil layer – varied from 10 to 50 ft (3-15m),
- soil density - varied as a pair of dry/wet density from 90/110 pcf (1450-1750 kg/m^3) to 120/140 pcf (1920-2240 kg/m^3), and
- depth of water table – varied from 2 to 20 ft (1-6m).

Only one parameter at a time was changed from the default values, and the interaction between two or more varying parameters was not evaluated. The sensitivity analysis is summarized in Figure 3, which plots percent change in each parameter versus resulting percent change in the critical $(N_1)60$ value.

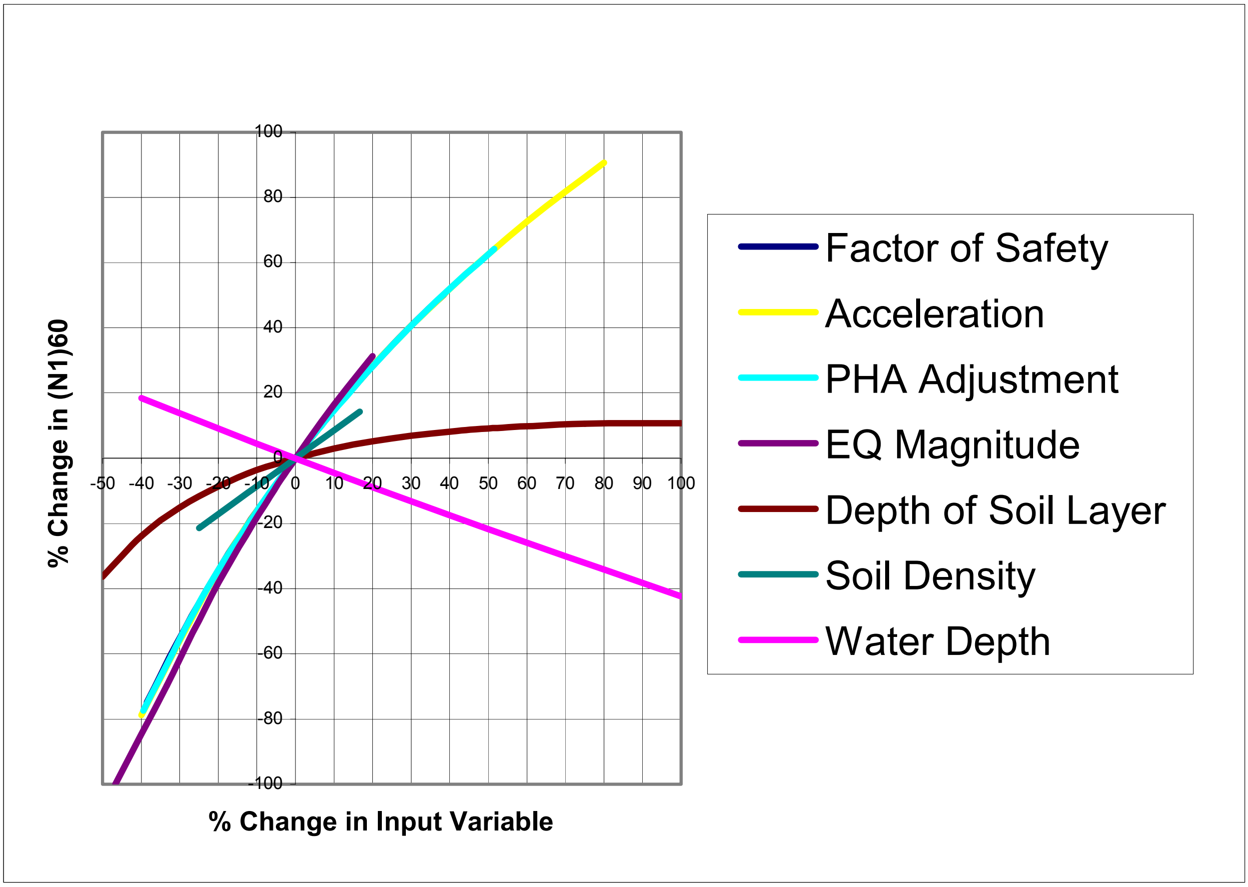

Figure 3. Sensitivity analysis of variables influencing liquefaction susceptibility. Input variables were modified and the effect was monitored on the critical $(N_1)60$ value below which liquefaction would occur.

Figure 3 shows that the least influential input parameter for the analysis is the depth of the soil layer, indicating that minor variations from the assumed depth of 20 ft (6m) will result in changes in critical $(N_1)60$ values of approximately 10 percent or less.

Water depth is roughly twice as important as soil depth, and soil density is roughly four times as important (although soil density only spans a short range of values).

The most important parameters, all of which are roughly six times as important as soil depth, are factor of safety, and earthquake acceleration and magnitude. For this study, the factor of safety and earthquake magnitude will be held constant as design parameters, but the effects of variation in earthquake acceleration will be closely analyzed.

Parametric Analysis

For "typical" soil profile and earthquake conditions outlined above, critical $(N_1)60$ values below which liquefaction is expected to occur are plotted in Figures 4 through 7. These figures are modifications to Figures 1 and 2, where the expected earthquake accelerations have been replaced by the critical $(N_1)60$ values for each case. As shown in the figures, critical $(N_1)60$ values are the highest for the soils with the lowest fines content, at the locations where highest accelerations are anticipated. Liquefaction susceptibility for 15 percent and 35 percent fines content soils under 10 percent PE accelerations (up to 9%g) is expected to be negligible, so figures for these conditions are not included.

The user should note that $(N_1)60$ values include depth and energy corrections, and will usually be substantially lower than field SPT values, especially for deep soils. For example, a field-measured SPT value for a sand unit at a depth of 20 ft (6m) may be 25, which is considered medium dense. Corrected for depth and hammer energy (using a donut hammer), the $(N_1)60$ value for this unit is 19, which would be susceptible to liquefaction throughout a large portion of western Colorado (Figure 5).

Figures 4 through 7 can still be useful if the "typical" soil conditions are changed. Based on inspection of the sensitivity analysis data in Figure 3, the following adjustments to critical $(N_1)60$ values should be made:

- for every 10 percent increase in water depth, decrease $(N_1)60$ by approximately 4.5 percent (similar, opposite changes are caused by decrease in water depth),
- for every 10 percent increase in soil density, increase $(N_1)60$ by approximately 9 percent (similar, opposite changes are caused by decrease in soil density),
- for every 10 percent increase in soil depth, increase $(N_1)60$ by approximately 3 percent, but
- for every 10 percent decrease in soil depth, decrease $(N_1)60$ by approximately 5 percent.

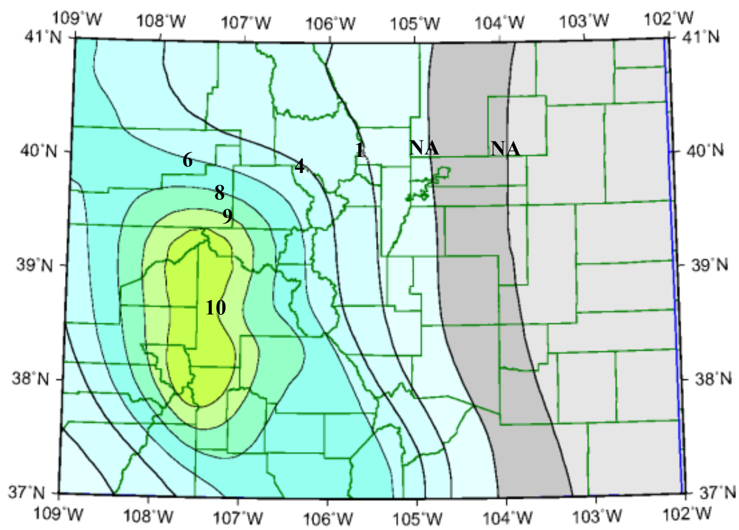

Figure 4. Liquefaction susceptibility related to earthquake accelerations generated by a 10% Probability of Exceedance in 50 years event. Numbers shown on contours are critical $(N_1)60$ values below which liquefaction of saturated, non-cohesive soils (5% fines or less) is expected. Soils with 15 to 35% fines are not expected to liquefy under this size earthquake event. Base map is from USGS (2003).

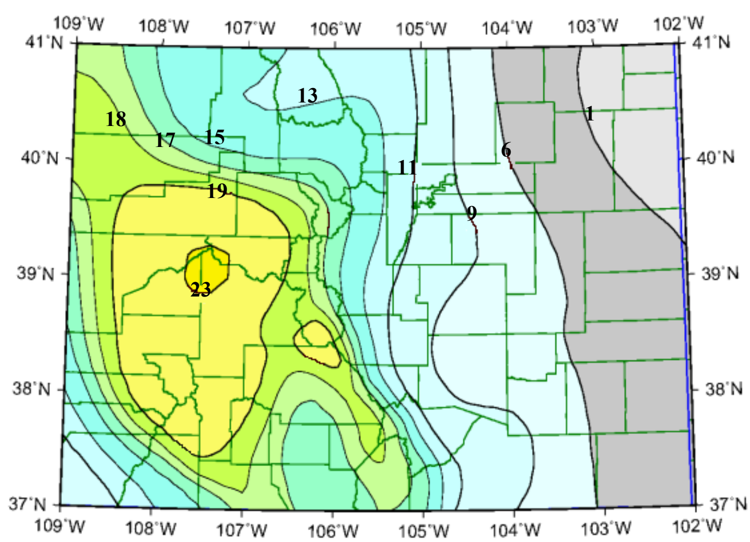

Figure 5. Liquefaction susceptibility related to earthquake accelerations generated by a 2% Probability of Exceedance in 50 years event. Numbers shown on contours are critical $(N_1)60$ values below which liquefaction of saturated, non-cohesive soils (5% fines or less) is expected. Base map is from USGS (2003).

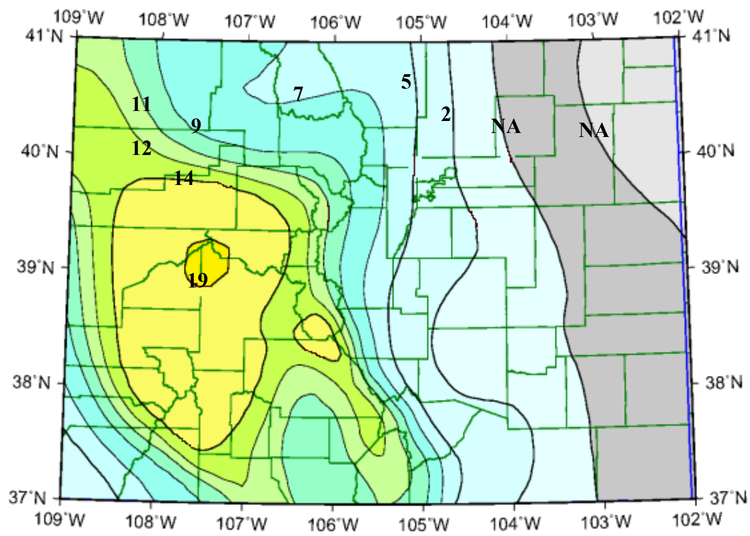

Figure 6. Liquefaction susceptibility related to earthquake accelerations generated by a <u>2% Probability of Exceedance in 50 years</u> event. Numbers shown on contours are critical $(N_1)60$ values below which liquefaction of saturated, non-cohesive soils (<u>15% fines</u>) is expected. Base map is from USGS (2003).

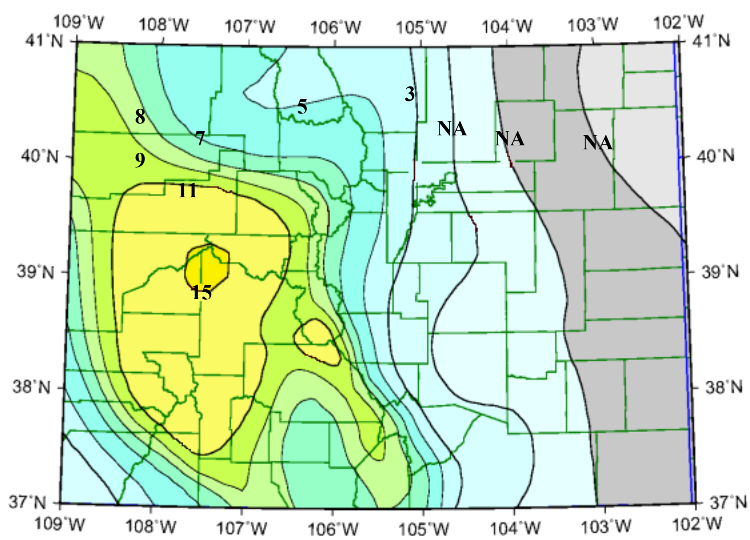

Figure 7. Liquefaction susceptibility related to earthquake accelerations generated by a <u>2% Probability of Exceedance in 50 years</u> event. Numbers shown on contours are critical $(N_1)60$ values below which liquefaction of saturated, non-cohesive soils (<u>35% fines</u>) is expected. Base map is from USGS (2003).

Soils along transportation routes that are expected to meet the conditions required for liquefaction include young sand, silt, and gravel deposits in river valleys, and basin fill alluvium in the San Luis Valley. Based on the susceptibility maps produced as Figures 4 through 7, the most sensitive areas are in the central western part of the state, including the river valleys of the Arkansas, Eagle, Colorado, Gunnison, Uncompahgre, Animas, Los Pinos, and Upper Rio Grande. Potentially vulnerable highways include Interstate 70; U.S. Highways 24, 50, 160, 285, 550; and numerous State Highways. Bridge crossings of alluvial beds and portions of the roadway placed on alluvium are expected to be the most susceptible to liquefaction. Potentially affected communities include Delta, Montrose, Ridgway, Durango, Gunnison, Creede, Del Norte, and the San Luis Valley (particularly the northern and southern ends).

LANDSLIDE SUSCEPTIBILITY

Slope movement in response to earthquake shaking is often modeled as the change in stability due to horizontal accelerations induced by the earthquake. Keefer (1984) suggests that there appears to be a threshold earthquake magnitude of approximately 4.0 to be capable of generating the acceleration magnitude and duration necessary to cause slope movement. Nevertheless, small earthquakes of long duration or larger earthquakes of short duration have the potential to induce slope movement that could damage or block roadways.

The analysis below was conducted to gauge the potential impacts of anticipated earthquake accelerations in Colorado on a variety of slope configurations and soil materials. No site-specific analyses were conducted, but the likely range of slope angles and soil strengths was assessed in order to identify the most susceptible materials and locations.

Methods

As was done for liquefaction analysis, slope stability was evaluated for "typical" soil conditions, for a range of slope profiles. The "typical" slope was assumed to consist of a layer of residual soil over bedrock, with both sloping at an angle of 25°. The default soil was considered to have dry density of 115 pounds per cubic ft (pcf) (1850 kg/m^3), wet density of 125 pcf (2000 kg/m^3), cohesion of 600 pounds per square ft (psf) (29 kPa), and internal friction of 20°. Bedrock was assumed to have dry density of 150 pcf (2400 kg/m^3), wet density of 155 pcf (2480 kg/m^3), cohesion of 3000 psf (144 kPa), and internal friction of 45°. The ground-water table was assumed to be located in the center of the soil layer (Figure 8).

Two different scale slopes were evaluated, as shown on Figure 8. The small slope was 20 ft (6m) long with a six-ft (2m) thick layer of residual soil, and the large slope was 200 ft (60m) long with a 30-ft (9m) thick layer of soil. The small slope was evaluated only for rotational failure, using a factor of safety of 1, and using the peak acceleration (from USGS, 2003) as the design acceleration. The large slope was evaluated for both rotational and translational failure, using a factor of safety of 1.1, and setting the design acceleration at 50 percent of the peak acceleration in order to better represent sustained accelerations during the earthquake (the factor of safety and design accelerations were selected based on recommendations and summaries of other analyses presented in CDMG, 1997; Kramer, 1996; and Day, 2002).

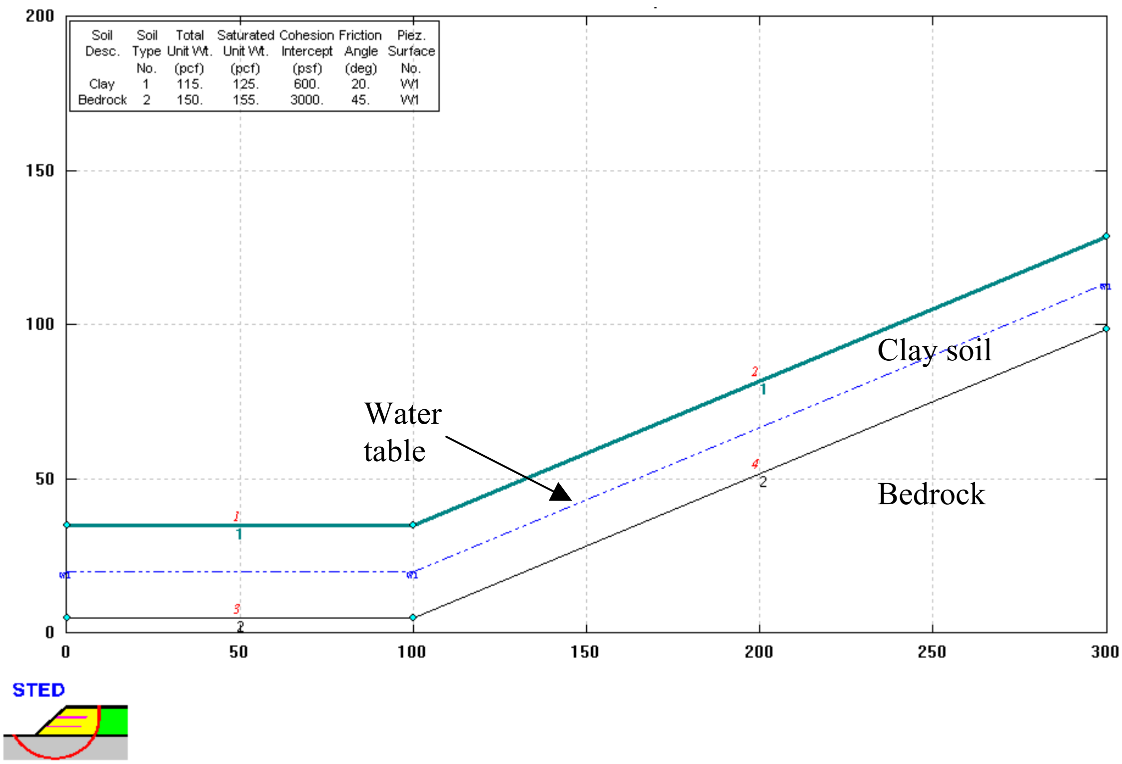

Figure 8. Typical slope profile analyzed for slope stability. The scale shown is for analysis of large slopes (200 ft or 60m long). Small slopes were also analyzed at 1/10 the scale shown.

Limit equilibrium analysis of slope stability was conducted using the computer program STABL6H (with pre- and post-processor STEDwin 2.33). Rotational failure surfaces were analyzed using the Simplified Bishop Method and translational failure surfaces were analyzed using the Simplified Janbu method.

Sensitivity Analysis

A sensitivity analysis was conducted with the same goals as for the liquefaction sensitivity analysis: to identify to importance of each input parameter in the overall analysis, and to quantify the change in factor of safety resulting from an incremental change in each parameter. The parameters to be changed and their range of variation are as follows:

- ground-water depth – varied in five increments from the top of bedrock (depth to water = soil thickness) to the top of the soil unit (depth to water = 0),
- soil cohesion – varied from 0 to 3000 psf (0-144 kPa),
- internal friction – varied from 5 to 40°,
- dry and wet density – varied as a pair of dry/wet density, from 105/115 pcf (1680-1840 kg/m^3) to 125/135 pcf (2000-2160 kg/m^3),
- slope angle – varied from 10 to 40°,
- earthquake acceleration – varied from 0 to 30%g, and

- soil thickness – varied from 3 to 60 ft (1 to 18m) for large slope only.

Only one parameter at a time was changed from the default values, and the interaction between two or more parameters was not evaluated. The analysis is summarized in Figure 9 (small slopes) and Figure 10 (large slopes). Only minor differences were noted between rotational and translational failure surfaces, and Figure 10 adequately represents both failure modes for large slopes.

Figure 9 shows that for small slopes, the depth to ground water had the smallest impact of the variables tested, with approximately 1 percent change in factor of safety for every 10 percent change in ground-water depth. Friction angle is roughly twice as important as ground water, and cohesion is roughly eight times as important as ground water (and four times as important as friction angle). Soil density is roughly eight times as important as ground-water level, although it spans only a short range of values. Slope angle is 10 times as important as ground-water level and earthquake acceleration is roughly 15 times as important.

For large slopes (Figure 10), ground water again has the smallest impact, with similar effect on the factor of safety (approximately 1 to 1.5 percent change for every 10 percent change in ground-water depth). Soil thickness, soil density, slope angle, cohesion, and friction angle are all roughly three to four times as influential as ground-water level. The notable difference from small slopes is that the friction angle is now as important as the cohesion. Earthquake acceleration again is the most important factor, with roughly 10 times the influence of ground-water levels.

Based on the sensitivity analysis, soil density only varies over a short range, and soil thickness and ground-water depth have smaller impacts than other variables, so these three parameters will not be varied from the default values in the parametric analysis. The effects of cohesion, friction angle, slope angle, and earthquake acceleration will be analyzed below.

Parametric Analysis

To assess the response of a range of slope configurations, soil strengths, and earthquake loads, a series of analyses were completed to systematically develop slope stability charts. To begin, friction angle was fixed at 20° and earthquake acceleration and slope angle were varied through the same range as in the sensitivity analysis. For each set of conditions, the cohesion was adjusted until the target factor of safety was achieved (1.1 for large slopes and 1.0 for small slopes, as described above).

These cohesion values (and associated accelerations and slope angles) were grouped into ranges of 50 psf (2.4 kPa) for small slopes and 200 psf (10 kPa) for large slopes. These divisions are reflected in Tables 1 and 2 and Figures 11 and 12. Next, representative values of cohesion, acceleration, and slope angle were chosen for each range as starting values, and the change in critical cohesion was then calculated for a range of friction angle values (10 to 35°). This second

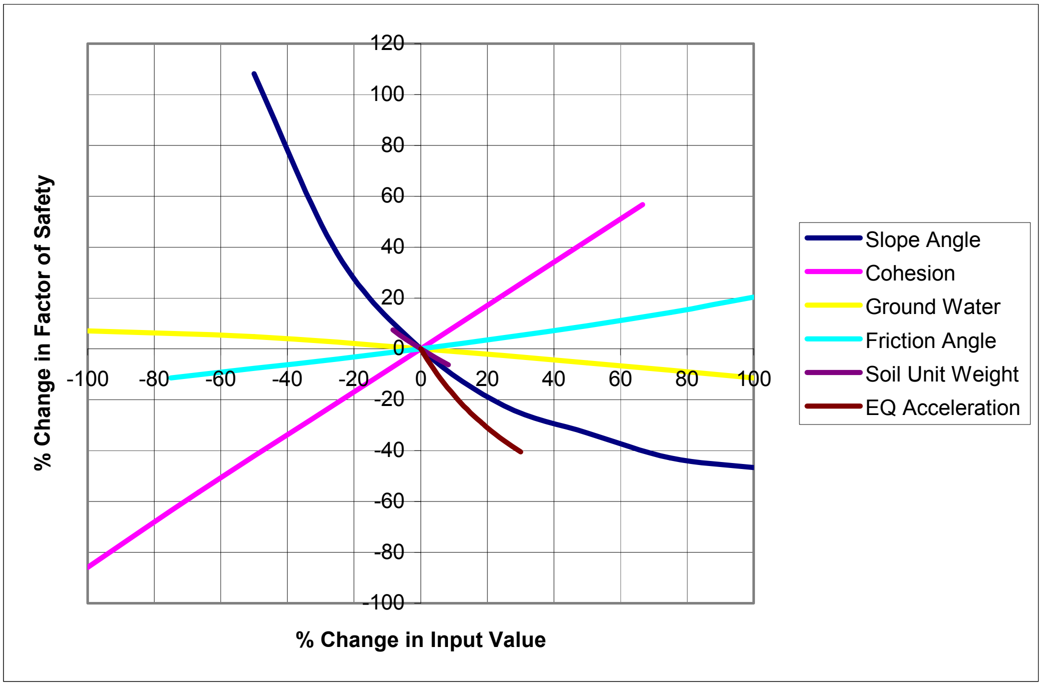

Figure 9. Sensitivity analysis of variables influencing stability for small slopes (< 100 cubic yds or 75 cubic m). Input variables were modified and the effect on factor of safety against sliding was monitored.

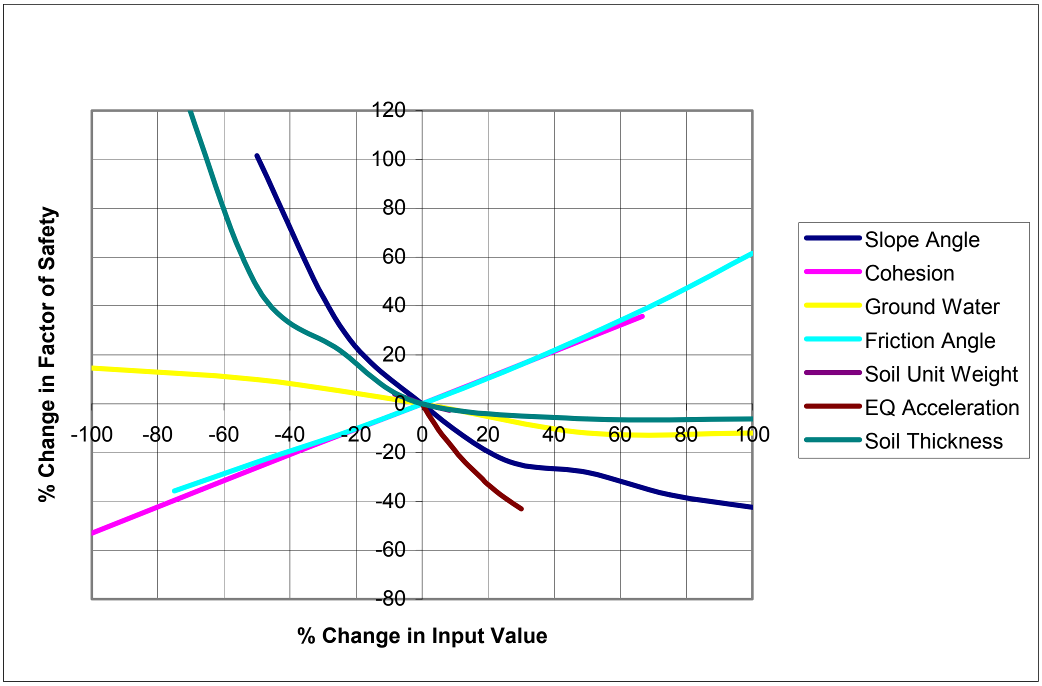

Figure 10. Sensitivity analysis of variables influencing stability for large slopes (> 10,000 cubic yds or 7500 cubic m). Input variables were modified and the effect on factor of safety against sliding was monitored.

Table 1. Summary of Parametric Analysis for Stability of Small Landslides (<100 cy or 75m^3)

Letters in the table show the proper slope stability graph to use based on the expected slope angle and earthquake acceleration. Conditions assigned the letter "A" are expected to be stable for soils with nominal strength (cohesion greater than 50 psf or 2.4 kPa).

Earthquake Acceleration (in %g)	Slope Angle (in degrees) 10	15	20	25	30	35	40
0	A	A	A	A	A	B	C
2	A	A	A	A	A	B	C
4	A	A	A	A	B	B	C
6	A	A	A	A	B	B	C
8	A	A	A	A	B	B	C
10	A	A	A	A	B	C	C
12	A	A	A	B	B	C	C
14	A	A	B	B	B	C	C
16	A	A	B	B	B	C	D
18	A	A	B	B	C	C	D
20	A	A	B	B	C	C	D
30	A	A	C	C	C	D	D

Table 2. Summary of Parametric Analysis for Stability of Large Landslides (>10,000 cy or 7500m^3)

Letters in the table show the proper slope stability graph to use based on the expected slope angle and earthquake acceleration. Conditions assigned the letter "E" are expected to be stable for soils with nominal strength (cohesion greater than 100 psf or 5 kPa)

Earthquake Acceleration (in %g)	Slope Angle (in degrees) 10	15	20	25	30	35	40
0	E	E	F	G	H	I	I
2	E	E	F	G	H	I	I
4	E	E	F	G	H	I	I
6	E	E	F	G	H	I	I
8	E	E	F	H	I	I	J
10	E	E	G	H	I	I	J
12	E	E	G	H	I	J	J
14	E	E	G	H	I	J	J
16	E	E	G	H	I	J	J
18	E	F	G	H	I	J	J
20	E	F	G	H	I	J	J
30	E	G	H	I	J	J	J

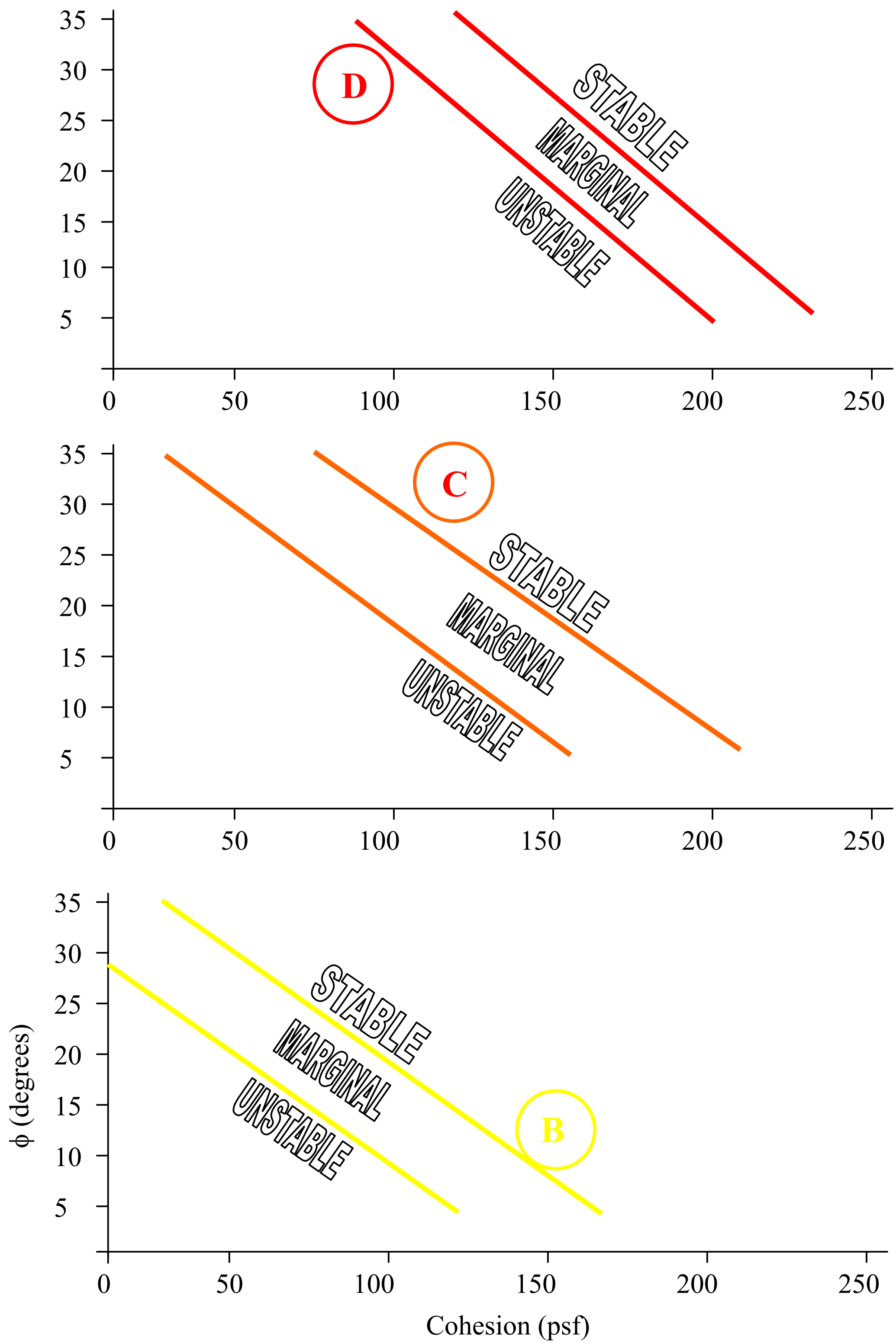

Figure 11. Soil strength influence on stability for small slopes (<100 cubic yds). To be used with Table 1. Conditions assigned to the letter "A" are expected to be stable.

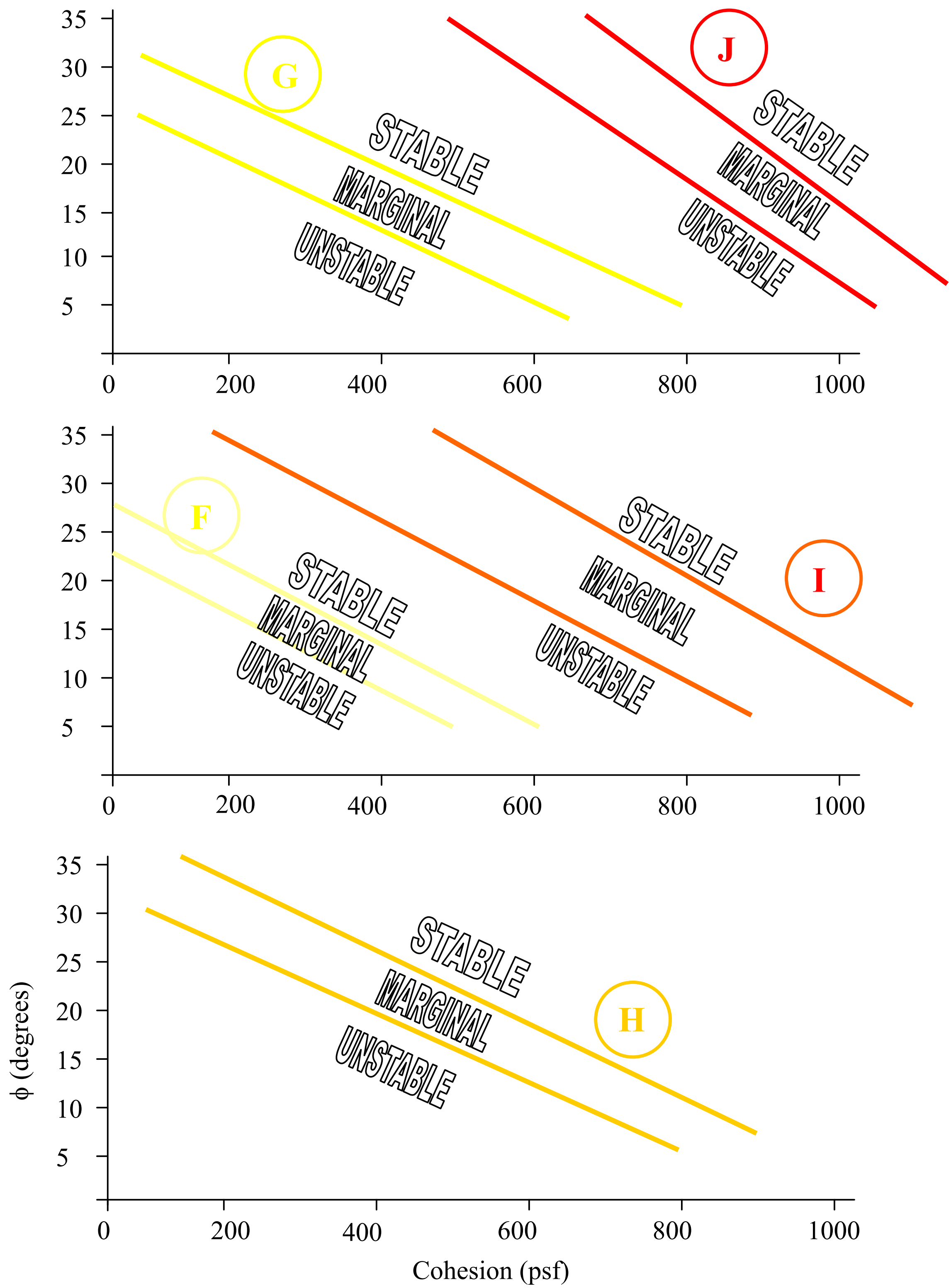

Figure 12. Soil strength influence on stability for large slopes (>10,000 cubic yds). To be used with Table 1. Conditions assigned to the letter "E" are expected to be stable.

set of analyses was used to set the width of the "Marginal" band on each graph in Figures 11 and 12. Finally, Tables 1 and 2 were developed to relate slope angle and earthquake acceleration to the appropriate strength curve on Figure 11 and 12.

In total, over 250 slope stability permutations were calculated to produce these figures and tables. Using them, one may select the slope angle of interest (or range of slope angles) and the appropriate expected earthquake acceleration, and then read the critical cohesion and friction values for slope stability. The figures are expected to apply to residual soils over bedrock (slope configuration is shown in Figure 8). Because of the variety of slope conditions represented, these values are not tied to state-wide earthquake acceleration maps, as was done for liquefaction susceptibility, but the user can read anticipated earthquake accelerations for specific locations off Figures 1 or 2, and then find the critical soil conditions for different slope angles from Tables 1 and 2 and Figures 11 and 12.

The ability to vary parameters from their default values and adjust the final result is not as straightforward as it was for the liquefaction analysis. However, the user is directed to the sensitivity analysis graphs (Figures 9 and 10) to assist in their evaluation of the influence of changing certain parameters. The sensitivity and parametric analyses are not intended to replace site-specific analyses, but to identify the ranges of soil and slope conditions expected to be vulnerable to earthquake-induced slope movement.

Vulnerability to earthquake-induced landslides increases with increasing earthquake acceleration, increasing slope, and decreasing soil strength properties. As for liquefaction, the most vulnerable areas are expected to be in the central western part of the state. The most vulnerable slope materials are expected to be saturated clayey soils with low to moderate strength, such as residual soils developed over fine-grained sedimentary bedrock, or thick alluvial or glacial sediments.

CONCLUSIONS

Based on the analyses presented above, the following conclusions are offered pertaining to earthquake-induced liquefaction and landslides in Colorado:

- Both processes will be most pronounced in the western mountains and plateaus, where anticipated earthquake accelerations are the highest.
- Only very loose sands and silts (corrected SPT values less than 5 to 10) are expected to liquefy following a 10 percent PE earthquake.
- Following a 2 percent PE earthquake, denser soils (corrected SPT values less than 10 to 23 for clean sands and silts, or less than 5 to 15 for soils with significant fines content) will experience liquefaction.
- Small landslides (less than 100 cubic yds or 75 cubic m) are expected to occur in steep slopes (30 to 40°) with weak soils (cohesion less than 100 to 200 psf or 5 to 10 kPa, friction angle less than approximately 20°), with earthquake accelerations on the order of 10%g. Specific conditions may be evaluated using a set of graphs and tables.

- Large landslides (greater than 10,000 cubic yds or 7500 cubic m) are expected to occur in shallower slope angles, with deeper and stronger soils, and at lower earthquake accelerations than for small slopes. Specific conditions may also be evaluated using design graphs and tables.

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INNOVATION BY TEAMWORK ENGINEERING GEOLOGY OF INTERSTATE 70 THROUGH GLENWOOD CANYON

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Key Terms: Glenwood Canyon, Interstate 70, environmental sensitivity, innovation, tunnels, foundations, rock slope engineering, bridges

ABSTRACT

The design and construction of Interstate 70 through Glenwood Canyon, Colorado provided a challenge that tested the skills and ingenuity of the project team, especially the engineering geologists and geotechnical engineers. Difficult and unusual foundation conditions, rockfall hazards, need for tunnels, and the demand to protect the environment while maintaining the interstate traffic were just some of the natural and man-made obstacles confronted.

The Glenwood Canyon design team met these challenges head-on through a strong multi-disciplinary team approach. This teamwork provided a new model for the resolution of difficult space constraints, in which the geologists and geotechnical engineers played an important role.

With final design beginning in 1980, and construction starting shortly thereafter, resolution of foundation design issues became a critical item on the project timeline. Foundation engineering challenges ranged from the unexpected existence of a gray clay layer at the canyon's east end, to the presence of significant deep voids in buried talus layers at the west end. Resolution of these challenges in a timely, environmentally sensitive manner required strong innovation and "out of the box " thinking on the part of the engineers and geologists. Foundation solutions included ground improvement, drilled and mined shafts, conventional piling, predrilled and blasted piling, and the development of retaining wall systems that could tolerate significant total and differential settlements.

Rock slope stability and rockfall mitigation was an important consideration in the development of highway alignment solutions. Rock slope engineering in Colorado (and around the world) benefited greatly by the development of the Colorado Rockfall Simulation Program (CRSP). Rock excavations were designed to minimize the probability of dangerous rockfall events and provide a natural appearance to the man-made slopes. The Glenwood Canyon I-70 project pioneered technical and contractual techniques to allow the economical construction of sculpted and stained rock cuts.

The demand to protect the environment and create an auto-free zone in the ever-popular Hanging Lake area required the development of many highway alignment alternatives. With the tightest natural curvature of the canyon in this area, a tunneling solution was chosen to be the best alternative to meet the goals of this project segment. Subsurface exploration, geotechnical engineering, and tunnel design proceeded over 5 years before the Hanging Lake tunnel construction began. The development of an innovative basis for rock reinforcement design opened the door for the first highway tunnels in America to be designed and constructed with rock reinforcement as a permanent means of support. This was accomplished using rock dowels and robotically applied steel fiber reinforced shotcrete. Unique contract administration techniques were used. The Hanging Lake tunnels were finished one year ahead of schedule, and under the original budget

Innovation was not limited to the tunnel structures. To meet the challenge of constructing a 4-lane highway within the area disturbed by the construction of Highway 6 in the 1930's, the design team developed a terraced alignment concept. This concept included continuous retaining wall systems to create the platforms for the eastbound and westbound lanes. To fit the highway into the limited space available, the roadway had to be cantilevered. This required the use of post-tensioned concrete slabs, which increased foundation loads. Precast concrete wall elements, tied to a shallow foundation minimized the construction traffic impacts and provided an elegant architectural solution.

The successful completion of Interstate 70 through Glenwood Canyon set the standard for highway design and construction through difficult environmentally sensitive terrain. Engineering geologists and geotechnical engineers were given a significant role on the design/construction team. This approach insured the most successful solutions by innovation through teamwork.

INTRODUCTION AND SETTING

Located about 150 mi east of Denver on Colorado's western slope, Glenwood Canyon is a 12-mi long scenic canyon of the Colorado River (Figure 1). This 2000 ft deep gorge was carved by the Colorado River as it eroded into the southern flank of the White River uplift. The down-cutting was accelerated by glacier melts during the Pleistocene Era. After thousands of feet of down-cutting, spectacular exposures of the Precambrian to mid Paleozoic formations occurred. Of special note are the cliffs of Cambrian Sawatch Quartzite overlying the Precambrian metamorphic and granitic basement rock.

Glenwood Canyon has served as a transportation corridor for over one hundred years. Ancient people were unable to use the canyon as an east-west corridor, as the sheer cliffs went straight down to the river. With the invention and use of modern explosives to blast a bench, the railroad was first built in 1887 at the base of the cliffs on the south river bank. A primitive wagon road was established on the north river bank in the early 1900's. This road was improved in the 1930's to a narrow two-lane highway (US highway 6) (Figure 2), which for many years served as the principal ground transportation link from the Colorado front slope to the western regions of the nation. The two-lane highway through Glenwood Canyon had a poor safety record. In 1979, the injury rate from traffic accidents was 1.5 times the rate for rural primary highways in Colorado. The most dangerous accidents consisted of head-on collisions and opposite direction

Figure 1. Glenwood Canyon

Figure 2. Old Highway 6 alignment. Note the over-steepened riverbank and uphill cuts.

sideswipes. The dark, narrow, roadway, combined with the proximity of the Colorado River and steep rock walls, contributed to the severity of the accidents. As the route location process for I-70 west of Denver continued in the late 1950's and 1960's, environmental concerns over the potential impacts of constructing an interstate highway through this spectacular scenic resource were raised. In 1968, the Colorado State Legislature passed a joint House-Senate resolution that stated the highway through Glenwood Canyon be "so designed that… the wonder of human engineering will be tastefully blended with the wonders of nature." As the highway design team found out decades later, the wonders of nature included significant geologic challenges to construction of an aesthetic 4-lane highway.

THE PROJECT TEAM AND THE PROCESS

The sometimes-conflicting requirements of planning the design and construction of a major interstate highway through the Glenwood Canyon corridor demanded a different management approach. The Colorado Department of Transportation (CDOT, then Highways) understood the need for a strong multi-disciplinary team approach. The approach taken was based on strong technical representation of disciplines led by a CDOT project manager dedicated full-time to the interstate corridor project. This teamwork provided a new model for the resolution of difficult space constraints in which the project geologists and geotechnical engineers played an important role. The team was composed of CDOT staff, consultants, and Colorado Geological Survey (CGS) staff dedicated full-time to the Glenwood Canyon I-70 project.

The project team's responsibilities, under the leadership of the CDOT project manager, were to identify the constraints, formulate methods for mitigation of adverse environmental impacts, develop plans for implementation, and manage the construction to meet all commitments. One of the very important roles of the project team was to win the confidence of those opposed to construction of an interstate highway through Glenwood Canyon.

To move the project out of a seemingly endless loop of preliminary design analysis, and into final design and construction, a two-stage fine-tuning process was developed (Trapani, et al. 1983). This process used the 1978 Design Report as a starting point, and through an intensive series of field reviews, subsurface explorations, and matrix analysis, "fine-tuned" alignments. Only after review of the fine-tuned alignment by 1) the designers responsible for the original preliminary design, 2) the CDOT Executive Director, 3) a Citizen's Advisory Committee (CAC) and 4) the Technical Review Group (TRG), could a project segment advance into final design, and ultimately construction.

OVERVIEW OF GEOLOGIC CHALLENGES

Initial reconnaissance of surface geology created the expectation of geomorphology consistent with that of a traditional mountain river valley. However, as subsurface exploration proceeded, it was noted that thick deposits of interfingering and mixed alluvial and colluvial sediments had aggraded in the canyon. Two startling discoveries were made. First, a deposit of clay, silt, and fine sand that ranged from 30 to 60 ft thick was discovered below the riverbed for the entire eastern half of the canyon. This deposit became known as the "Gray Layer." A second discovery was that many areas within the canyon had Holocene Epoch sediments on the order of 200 ft thick overlying bedrock. In the early 1980's when the first exploratory drilling occurred, the discovery of these two geologic features shattered the engineers' and geologists' intuitive model. Subsurface exploration (which included exploratory drilling and geophysical methods) proceeded in haste to support the alignment fine-tuning effort. Additional discoveries of thick aprons of buried, void-ridden talus on the canyon sides, interfingered with alluvial deposits confounded efforts to use conventional foundation design and construction techniques. Other geologic hazards that were apparent from surface geology and highway maintenance records included rockfall, and to a lesser extent, debris flows, ice falls and avalanches. The geologic setting is reported in (Colorado School of Mines, 1984) and shown in Figure 3. These geologic challenges and hazards described above required strong innovation and teamwork to protect the traveling public from geohazards and produce acceptable foundation solutions. It was critical to the success of the project, as the foundation systems for the retaining walls and bridges that delineate the highway structure on Interstate 70 through Glenwood Canyon were critical, early-action project elements.

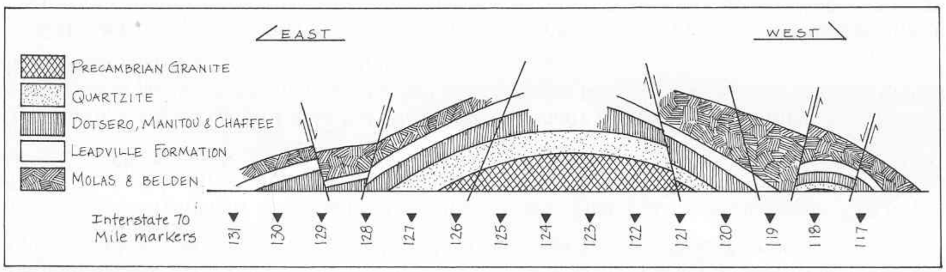

Figure 3. Geologic cross-section, from McGregor, (1992).

THE UNIQUE ROLE OF THE GEOLOGISTS AND GEOTECHNICAL ENGINEERS

The story of Interstate 70 through Glenwood Canyon is not all about rock, soil, steel, and concrete. The success of the project is due, in great part, to the attitude, skill, and focus of the personnel who worked on the project. Everyone involved was inspired by the beauty and grandeur of the Glenwood Canyon. This inspiration was manifested in an intense energy and desire to view challenges as "letters addressed" to the team, that demanded an answer. Most of these metaphorical "letters" were written by Mother Nature herself, in the form of geologic challenges.

The engineering geologists and geotechnical engineers held a role of unprecedented importance on the design/construction team. Never before had a highway project required such a diverse multi-disciplinary group. Blending an extremely talented group of engineers, architects, and scientists into a productive results-oriented team was not without its challenges. The engineering geologists and geotechnical engineers enjoyed significant responsibility in project decision-making, and were given the commensurate authority. This project is unlike typical highway projects, where geologic challenges could be resolved by simply moving the alignment. The team on Interstate 70 through Glenwood Canyon had to confront these challenges head-on, through a combination of teamwork, talent, and a profound respect and understanding for every discipline and their views.

While this philosophy is not applicable to every project, it still serves as a valid model to successfully complete the design and construction of facilities through difficult terrain. In more recent times, this management team model with strong representation by engineering geologists and geotechnical engineers has been applied with some success to other corridor projects in Colorado. As more of these tough projects are undertaken in the future, the authors suggest that this multi-disciplinary team model, with a substantial role and responsibility for the engineering geologists and geotechnical engineers, be considered.

DESIGN AND CONSTRUCTION OF RETAINING WALL FOUNDATIONS

One of the earliest design commitments for Interstate 70 through Glenwood Canyon was to "tread lightly" on the natural terrain (Guenther, 1993). Highway 6 construction, and the subsequent maintenance practice of cleaning the uphill rockfall catchment ditch and dumping the spoils on the riverside embankment, created a disturbed area above and below the highway. Early in the design process, it was envisioned to use a terraced alignment of continuous retaining walls to locate a 4-lane highway within that disturbed area (Figure 4). Not only did this alignment concept allow optimum use of the existing disturbance, it had several other benefits. First, it allowed the "healing" of the unstable uphill cut slopes through re-grading and landscaping. This benefit was more than just visual, as it eliminated the majority of the rockfall accidents in the canyon by stopping the potential for small rocks to roll out of the cut slope into the traveled way. Another benefit of the terracing was to allow the re-grading and landscaping of the riverbank that improved: 1) the visual appearance of the riverbank, 2) provided scour protection for retaining wall footings, and 3) created a level platform for the construction of the continuous recreation path.

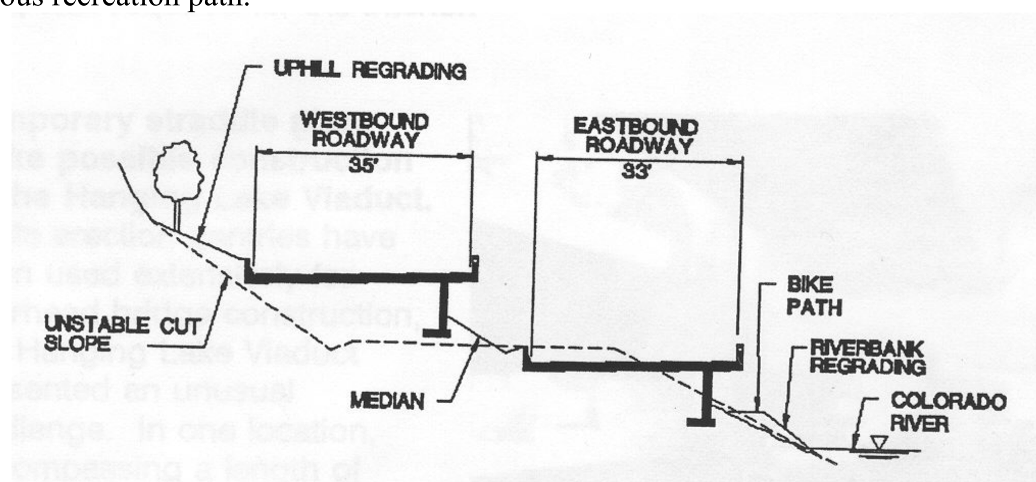

Figure 4. Typical section, terraced alignment, from DMJM, (1993).

This continuous retaining wall system created platforms for the eastbound and westbound lanes. In some cases, the roadway had to be cantilevered over the wall edge, using a six-ft overhang.

This element necessitated the use of a post-tensioned concrete slab. The bearing of this slab on the precast concrete twin-tee retaining panel created very high bearing loads, which were a challenge to resist using a shallow foundation. For the majority of the length of the 15 mi of retaining walls built, geotechnical engineers were able to assign allowable bearing values of up to 3 tons per square foot for alluvial gravel deposits. This allowed for the design and construction of economical spread footings for walls used on the terraced alignment. The terraced alignment also required significant numbers of tieback and tiedown anchors for the retaining wall structures (Figure 5). Due to the variability of the subsurface bedrock profile, and the difficulty in predicting its location, rock anchor technology was not applicable in most cases of retaining wall support systems. Constructing walls using permanent soil anchors was the only viable solution.

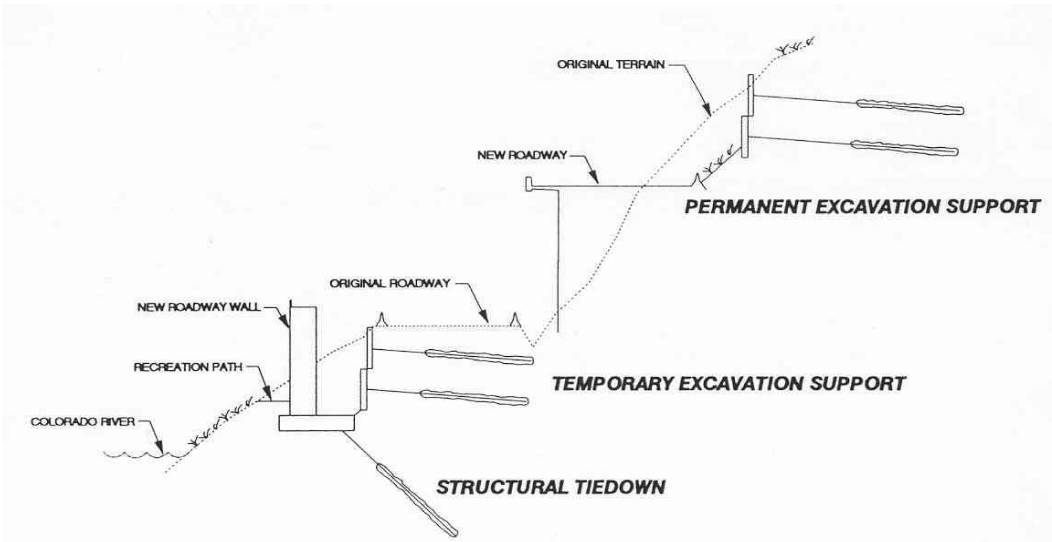

Figure 5. Anchor/wall configuration, from Kracum, (1993).

When the final design and construction began in the early 1980's, very little information was available on transportation project-related soil and rock anchor installation. Permanent soil anchors in buried talus slopes had not yet been documented anywhere. Initial installation utilized multiple anchors with each precast wall element. These early designs were very conservative, and designed in accordance with Post Tensioning Institute (PTI) recommendations (PTI 1980). Contract requirements included full size test anchors prior to the installation of production anchors. This not only verified design assumptions, but gave all parties involved an idea of the constructability of the anchors at a given location. This testing was paid for separately from production anchors. For the permanent anchors, pay quantities were based on the unit length of anchor installed, which included all materials, labor, and equipment necessary to complete all drilling, tendon installation, grouting and proof testing.

The early anchor installations had substantial overruns in grout, even using the best construction practices (Kracum, et al. 1993). This overrun could be attributed to migration of the grout material through the soil. Concerns over lost encapsulation due to this grout migration were minimized by visual verification that the bond zone was completely grouted during primary grouting. However, these grout overruns and related schedule impacts were not acceptable to the contractor, and CDOT agreed to share the extra costs. This provided a valuable lesson for both the owner and the contractor in future installations. It should be noted that, in spite of the contract administration challenges created by the method of payment for the anchors, it was successfully demonstrated that soil anchors could be installed in buried talus/soil conditions, as shown in Figure 6 for example.

This early experience showed that much higher anchor capacities were being obtained than originally anticipated. This increased confidence led to elimination of the 4 anchor per panel scheme in favor of a single anchor per panel design, which up until now had only been allowed for temporary excavation support.

Figure 6. Tie-back wall in talus at Shoshone Power Plant.

As more experience was gained with the single anchor panel system, design modifications continued to enhance the system. For example, the use of grout containment devices became more widespread. Changes in the project specifications and contract administration techniques allowed contracting parties more flexibility. While still specifying anchor location and length, performance criteria were given for drilling, anchor components, tendon fabrication, corrosion protection, grouting and grout containment devices. The later generation specifications required CDOT approval of an experienced anchor contractor to perform the work. In addition, historic "grout take" data for previous installations was made available to all prospective bidders. Ultimately, nearly 40,000 lineal ft (12,000 m) of ground anchors were installed on the project.

DESIGN AND CONSTRUCTION OF BRIDGE FOUNDATIONS

Bridges and viaducts were used extensively on the project. Forty bridges, totaling a combined length of over 6.2 mi (10 km) were designed and constructed. Many of the structures were required to span over natural and man-made features in the Canyon. Non-traditional situations demanded the use of bridges and viaducts for environmental protection and reclamation of damage caused by the construction of US Highway 6.

The extensive use of these bridge and viaduct structures presented special challenges to foundation designers. The geology of the canyon, in itself, was a significant challenge to foundation designers seeking to transfer loads to suitable bearing materials. In addition, the innovative use of precast segmental concrete bridge construction, using balanced cantilever techniques (Phipps, 1993; McNary and Harding, 1993) created significant loading on foundations during the construction process. The complex subsurface geology of the canyon, particularly for the long viaducts, created challenges that could only be resolved in the field. Elsen (1993) reported a dramatic case study of a deep foundation redesign.

One of the early construction projects in the West End area between No Name and Grizzly Creek utilized several bridges to span natural draws and minimize the use of retaining walls (Figure 7).

Figure 7. Bridges and walls at west end.

During the design of the project, subsurface investigations showed the presence of buried talus deposits. Where these deposits occurred under the footing, acceptable bearing values could not be assigned. In addition, the presence of large voids created the possibility of sudden, catastrophic settlements due to increased loads in the talus layer. To resolve this geologic challenge, highway designers adjusted the alignments to lower the height of the walls. This reduced foundation loads through reduced wall heights. Next, a test program was initiated to evaluate grouting as an option to penetrate the buried talus layers. There were several ill-fated attempts to pump conventional thin cement grouts into the buried talus, with no recovery of these grouts due to the interconnected voids. The next idea was to design and test a system of compaction grouting. This technique proved very successful. Since confinement of grout at depth was an issue, initial grouting was accomplished using a low-slump sand/cement grout, injected under high pressure, to create a "curtain" of grouted talus rock around the perimeter of the future foundation element. The next step was to drill a pattern of holes (usually spaced every 4 ft) within the perimeter of the future footing, and pump more low-slump grout at very high pressures. A soil surcharge was used to prevent the ground surface from heaving. At the completion of the second phase of compaction grouting, the footing could be constructed. The compaction grouting proved to be a very successful solution for bridge piers located over buried talus formation and in loose river sands. It allowed a cost-effective foundation solution, which could be constructed using small, low-impact equipment.

Not only did the side-hill bridges founded over the buried talus deposits create a challenge to the innovation efforts of the design team, bridges founded in and directly adjacent to the Colorado River channel expanded that challenge. Excessive construction loading conditions, hydrodynamic forces on bridge piers, and local scour concerns were critical factors in the foundation design. While the Colorado River in Glenwood Canyon tends to be a statically stable river, with peak flows during spring run-off leveled by upstream diversions, subsurface investigations in and adjacent to the river channel indicated highly variable conditions. Up to 200 ft of alluvial or colluvial deposits over bedrock were encountered.

Constructibility concerns of these river bridge foundations also were raised. The presence of large boulders in the alluvial/colluvial deposits prevented the use of conventional steel H-piling. Also, the construction of Highway 6, using traditional cut and fill techniques, created tightly packed shot rock clusters buried in the river banks that challenged conventional pile-driving techniques.

In areas where steel H-piling presented an acceptable design solution, but subsurface investigations indicated subsurface boulders and man-made rock pockets, special techniques were developed. Contract documents allowed the use of pre-drilling and pre-blasting techniques, which consisted of first core drilling to near the planned pile-tip elevation. As the core drilling proceeded, drill logs indicated the vertical location of large boulders/shot rock clusters. The drill hole was loaded with explosives at the appropriate depths and detonated. This resulted in a pulverized column of material, through which H-piling with specially reinforced tips could be driven. Extensive use of dynamic pile-driving analyzers and full scale load testing (Figure 8) allowed the construction team to analyze the behavior of the steel piling during driving, to both ensure that the piling was not being damaged, and to verify the as-constructed capacity of the piling element.

Extensive use of caisson foundations occurred where subsurface conditions and foundation loads required them. While several bridges at the east end of the canyon were successfully founded on large diameter conventional drilled caissons, the Shoshone Power Plant bridges, on the eastbound mainline and off-ramp alignments required a more innovative solution.

The section of Glenwood Canyon at the Shoshone Power Plant represented a unique challenge. Large bridges were necessary to span the nearly 100-year-old power plant. In addition, an interchange was necessary to provide access from I-70 both for power plant personnel, and to whitewater enthusiasts wanting to utilize a planned raft/kayak put-in. The space was so constrained in this area that a fully directional interchange was not possible. Since most of the demand for access occurred from the west end of the canyon (Glenwood Springs), it was decided to construct an interchange with the off-ramp in the eastbound direction and the on-ramp on the westbound side, for return to Glenwood Springs. The eastbound main line and ramp bridges had

Figure 8. Piling load test.

substructure elements located at the edge of the river channel. Hydrodynamic forces and scour were very serious considerations facing the design team.

The project was let for bids with an innovative foundation design, using large diameter shafts. The bridge piers were founded on 7-ft (213.36 mm) diameter shafts. The number of shafts depended on the loadings on that particular pier. Pier loads varied as a result of the superstructure design. The contractor elected to use a highly specialized West German construction technique called the Hochstrasser-Weise system. The system consisted of a thick wall (3.14 in) steel casing equipped with a 1 ½ in hardened steel cutting shoe at the driving end. The barrel had a hydraulically operated swing arm attached to the top (surface) end (see Figure 9). The rotational force of the swing of the arm, combined with the weight of the barrel, penetrated the river deposit. As the barrel advanced, various tools were lowered into the barrel to break up and remove boulders. In some cases, pre-blasting was used. Material was excavated in a saturated condition using a clamshell and/or suction bailers lowered into the barrel. After the planned elevation for the bottom of the shaft was reached, concrete was placed and the positive head pressure was augmented with compressed air applied between the top of the wet concrete and a steel pressure cap. The hydraulic swing arm was actuated to break the adhesion between the casing and soil. With the help of a crawler crane, the barrel was extracted before the concrete had set.

The construction documents for the project contained tolerances for the final caisson location and plumbness. These tolerances were based on the use of a conventional auger drilling technique in homogenous soils. With the choice of the Hochstrasser-Weise system and the non-homogenous environment, these tolerances could not be met. As the steel casing advanced, it became impossible to correct for location and plumbness. A decision was made to evaluate each caisson using group pile analysis. To verify a defect-free installation, Stress Wave Propagation

Figure 9. Hochstrasser-Weisse caisson rig.

Testing (SWPT) was used. This testing is well documented in Hsieh, et al. (1988). In addition, a full-scale lateral loading test was performed. A total of 41 caissons were successfully installed with lengths ranging from 18.5 ft (5.5 m) to 41 ft (12.3 m).

The award-winning bridge structures that are such a key element of I-70 through Glenwood Canyon remain some of the most visually impressive edifices on the project (Figure 10). It is unfortunate that the best work of the engineering geologists and geotechnical engineers is buried under the surface.

DESIGN AND CONSTRUCTION OF TUNNELS

With the demand to protect the environment in Glenwood Canyon and the design intent to create an auto-free zone at the popular Hanging Lake area, tunneling alternatives were the preferred choices at two locations. At the Reverse Curve area, about 2 mi west of Bair Ranch, a 600’ long tunnel was constructed for the westbound lanes. This minimized the width of the highway platform to the point that the eastbound lane could be successfully fit as a surface alternative, without extensive rock cuts or river fills.

The Reverse Curve tunnel (Figure 11) was driven through the Sawatch Quartzite (a metamorphosed beach deposit), a 500-ft thick stratigraphic unit of the upper Cambrian age, which overlies Precambrian basement rocks. At the tunnel site, the formation dips gently about 10 degrees to the southeast as the result of uplift along the southeast plunging White River Plateau arch (Leeds, Hill and Jewett, 1981a).

Figure 10. Hanging Lake viaduct.

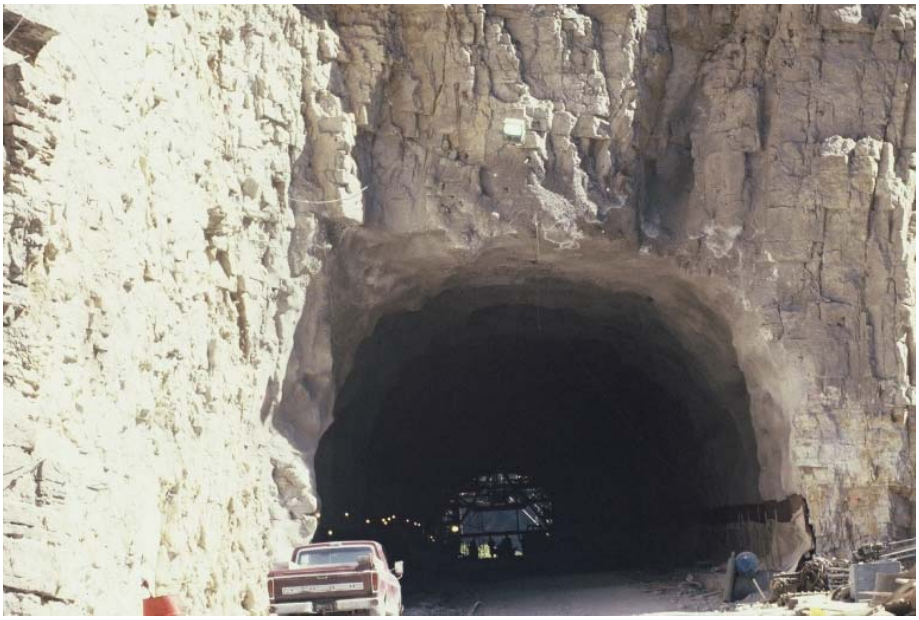

Figure 11. Reverse Curve Tunnel, west portal.

A major challenge to the design team came in the form of rock discontinuities in the Reverse Curve rock mass. Generally, the rocks at the site are very strong, and showed few signs of weathering and alteration. However, major open fractures, joints, and weak bedding plane partings governed the behavior of the rock mass.

Geotechnical concerns focused on maintaining the tunnel opening. The tunnel alignment created a narrow rock pillar that occurred at both the east and west portals, which demanded great care (including extensive instrumentation and blasting control) be taken with rock excavation at those locations.

Based on the above concerns, geotechnical investigations focused on mapping and evaluation of fractures and joints where potential loosening and fallout of rock blocks and wedges from the roof of the tunnel could take place. With these conditions anticipated by the design team, it became apparent that the Reverse Curve Tunnel could be supported with rock reinforcement and shotcrete.

The design team began an investigation of the methodology of tunnel support design. This was documented by (Parsons, Brinckerhoff, Quade and Douglas, Inc., 1981). Besides reporting on the traditional procedure for rock tunnel support/lining design, the paper proposed an rational analytic method for rock reinforcement design, using site-specific geotechnical information.

After review by the various entities involved in the approval of the tunnel support design method (most importantly the Federal Highway Administration), the proposed method was accepted. This led the way to the Glenwood Canyon tunnels being the first road tunnels in the United States having the permanent support system of rock reinforcement and steel fiber reinforced shotcrete. Steel ribs were not used on either the Reverse Curve single-bore or Hanging Lake twin-bores. The concrete lining is considered non-structural. All rock loads are being handled by the rock reinforcement and shotcrete installed as both temporary (during construction) support and permanent support.

The theoretical basis for this innovative tunnel support design procedure involves modeling the roof of the tunnel opening as a group of reinforced rock units (RRU). Each RRU consists of an individual rock bolt and the rock immediately surrounding and adjacent to it. The design intent is to reinforce the RRU's so they are stable relative to one another and act together as a structural member. For the Reverse Curve tunnel, the RRU's were analogous to "voussoirs" in a masonry arch. The roof (or tunnel crown) is modeled as a Voussoir Arch. Using one of the basic assumptions made by C.A. Coulomb (in a paper dated 1776) that there be no sliding between "voussoirs" or RRU's, roof behavior can be predicted (Lang, 1981a and b). This innovation in tunnel support design was a result of outstanding creative thinking by the tunnel geotechnical design engineers, with the guidance and support of the entire design team. Development of this design technique is one of the most powerful examples of innovation by teamwork on the Interstate 70 project through Glenwood Canyon.

The design of the Reverse Curve tunnel proceeded using rock reinforcement as permanent support. The construction was very successful, and the tunnel was completed in 1986. As the construction of the Reverse Curve tunnel proceeded, the design of the twin-bore 4,200-ft (1,220

m) long Hanging Lake tunnels continued. The main purpose of these tunnels at this site was to improve the highway alignment and create an auto-free zone in Glenwood Canyon through the Hanging Lake trailhead area.

The surface geotechnical investigations for the Hanging Lake tunnels began late in 1980. Initially, two horizontal exploratory holes were drilled from drill set-ups in the Cinnamon Creek area. Borehole HL-1 was intended to explore rock conditions west of Cinnamon Creek. The exploratory diamond core drilling was very challenging for this hole, as it progressed 650 ft into the rock, at a downward angle of about 1.5°. Because of squeezing and caving conditions, it was necessary to change core sizes twice, with the smallest core size at termination of drilling of BX (1.432 inch) (Leeds, Hill and Jewitt, 1981b).

While a vast amount of data was acquired from the HL-1, much of the core recovered (85% core recovery) was intensively fractured. This tended to give low values for the Rock Quality Designation (RQD), which indicated poor tunneling conditions. This low RQD value was of concern to the design team, as the surface geology of the pre-Cambrian metamorphic and igneous rocks indicated that higher RQD's should be expected. To resolve this question, it was decided to construct an exploratory tunnel.

The exploratory tunnel was driven both east and west from the Cinnamon Creek valley. It was excavated by conventional drilling and blast techniques. The tunnel cross-section was about 12 ft wide and 13 ft high, and was located in the center, top area of the future east bound tunnel alignment.

The exploratory tunnel construction began in November 1983, and was completed in early 1985 by a contractor whose bid ($2,140,812) was less than half of the engineer's estimate. As a result of data gleaned from the exploratory tunnel, it was determined that the rock mass conditions were much better than that indicated by borehole HL-1. It was theorized later that the low RQD's from the HL-1 borehole were due to mechanical breakage of the core during drilling and core recovery. The major portion of the rock mass exposed in the exploratory tunnel was unaltered (rock shows no discoloration, loss of strength, or any other effect of weathering) to slightly altered (rock is slightly discolored but not noticeably lower in strength than fresh rock). This was exciting information, as it indicated good tunneling conditions, and allowed the use of rock reinforcement and shotcrete (and the pioneering tunnel support design technique described earlier) for the final tunnel support. It also proved the value of the exploratory tunnel. Had the project been designed using only data from the boreholes, a very costly conservative tunnel support design would have been used. Based on anecdotal information, various tunneling experts believed that the exploratory tunnel ultimately saved tens of millions of dollars on the project by the availability of accurate geotechnical data.

With completion of the exploratory tunnel, the design team continued with final geotechnical engineering of the Hanging Lake tunnels. Three main rock types were noted, including quartz diorite, migmatitic and pegmatitic, and granitic sills and dikes (Woodward Clyde Consultants, 1988). For bidding purposes, three types of tunnel stabilization were called for. Type-1 support (which was 70% of the tunnel length) required 12 ft long bolts in a 6-ft by 6-ft pattern. Type 2 ground (21% of the tunnel length) required type 1 rock reinforcement, spiling (15 ft long #10 bars installed 30° above horizontal, at 4 ft centers), a 2-in layer of shotcrete and reduced round lengths. Type 3 ground (9% of the tunnel length, mostly through shear zones) required type 2 reinforcement and spiling, reduced round lengths and a second 2-in thick layer of shotcrete. Shotcrete was steel-fiber reinforced. All stabilization types required a multiple slash top heading and bench excavation sequence (Figure 12). No steel ribs or structural concrete were required for tunnel support.

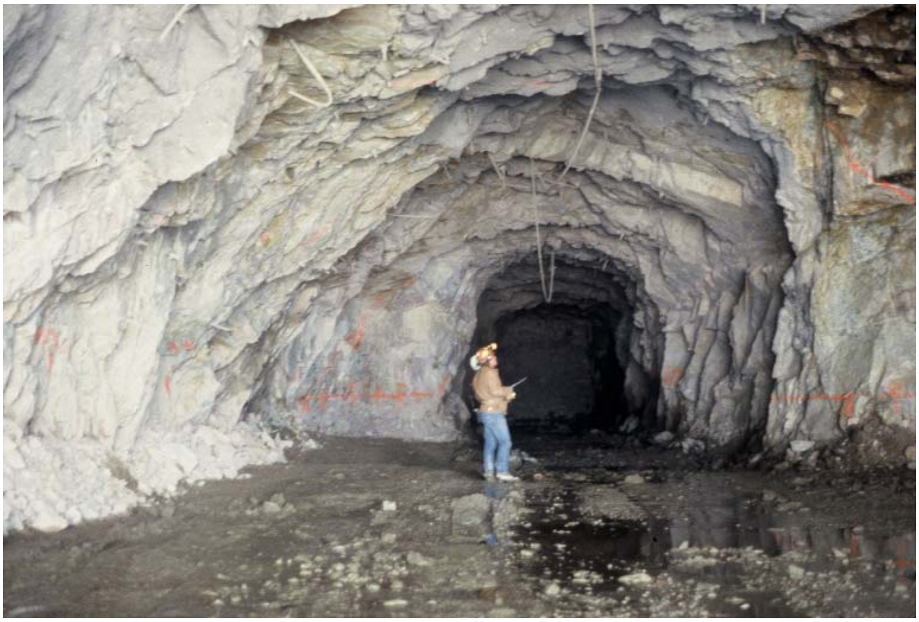

Figure 12. Multiple slash excavation, Hanging Lake Tunnel.

Overall, with the placement of the tunnel bores in the Pre-Cambrian metamorphic and igneous rock, and minimal flows of ground water, tunneling conditions were assumed to be quite favorable (Essex, et al., 1993). The presence of good rock for the tunnel excavation allowed for additional innovation in the contract administration techniques used for the Hanging Lake tunnel. It was decided to use an "Americanized" adaptation of the New Austrian Tunneling Method (NATM). NATM exploits the inherent strength of the virgin rock mass, and strives to minimize displacement soon after excavation (blasting). Rock reinforcement and shotcrete are installed as soon as safely possible after excavation, to form a safe, stable tunnel opening. By considering a systems approach to rock excavation and support, the adoption of an "Americanized" version of NATM was a strong innovation for tunneling in the United States. The "Americanization" of NATM focused on a partnership between the contractor's representatives and owner's representatives at the tunnel excavation face. Decisions on the support type were determined jointly by the contractor and the engineer. This put a large amount of responsibility (and authority) on the project's engineering geologists and geotechnical engineers. This innovative partnership approach worked very well in almost all cases. It was a rare and unusual occurrence for project management to be called in to facilitate the joint decisions by field staff. This is a strong tribute to the individuals employed by all parties, as technical skills became as important as people skills in such a challenging project environment.

The tunnel construction was split into 3 separate contracts. The main contract was for tunnel construction including excavation, support, tunnel lining and finishes, and paving. A second contract was let to furnish and install the 8 huge centrifugal fans to ventilate the tunnel. A third contract was let to furnish and install the tunnel control system, including the computer system, message signs, closed circuit TV, etc.

The main contract was bid in July 1989. Five companies bid the project, with the low bidder coming in $14 million under the engineer's estimate of $81.9 million dollars. The successful low bidder was a joint venture led by Frontier Kemper and Traylor Brothers of the United States, joined by Wayss and Freitage (Germany) and Beton and Monierbau (Austria). Excavation of the tunnel began in late 1989 Figure 13). The drill and blast work was completed in late November 1990. The early completion of the tunnel excavation (nearly 4 months ahead of schedule) allowed CDOT to work with bridge contractors at the east (Hanging Lake Viaduct) (Figure 14) and west (Shoshone Dam bridges) to accelerate the overall project completion.

Figure 13. Tunnel construction, Hanging Lake Tunnel.

The early completion of the tunnel can be attributed to several factors. First, the contractor team was well managed and highly organized. Second, the significant effort in subsurface investigation resulted in an accurate, complete set of construction plans that required significant instrumentation (Scotese, et al. 1999). This instrumentation improved confidence and allowed excavation to proceed in a timely manner. Third, the contractor used a new generation hydraulic rock drill (Hamrin, 1993) that greatly enhanced production rates. Finally, the demand for partnership decisions between the contractor and CDOT forces created a strong team mentality, with all parties sharing a common ground for success.

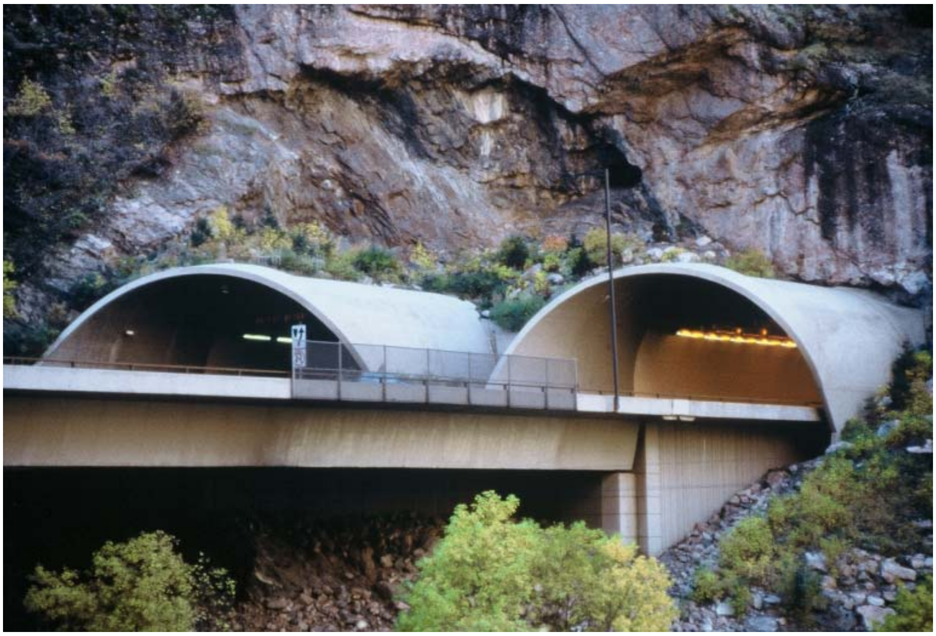

Figure 14. Completed east portals, Hanging Lake Tunnel.

The early completion, accomplished with no additional cost over the as-bid budget, make the Hanging Lake Tunnels one of the most successful vehicle tunnel projects accomplished to date in the United States. The innovative approaches taken by the design team, coupled with the strong skills of the project engineering geologists and geotechnical engineers and well-organized contractor team, clearly “raised the bar” for tunnel projects in the United States.

ROCK SLOPE ENGINEERING – BLENDING ART AND SCIENCE

After the construction of US6 through Glenwood Canyon in the 1930’s, the traveling public had a narrow two-lane roadway through this scenic gorge. Due to the construction methods of the time, uphill cuts excavated in the alluvial/colluvial material and blasted into bedrock cliffs tended to be over steepened. Over the years, these man-made cuts raveled, creating numerous small rockfalls. These rockfalls, while rarely actually striking a vehicle, required motorists to take evasive actions. On the narrow, two lane roadway the consequences of this evasive action by drivers resulted in dangerous head-on or opposite direction sideswipe accidents.

Early in the design of interstate 70 through Glenwood Canyon, the design team evaluated historic rockfall incidents through searches of accident records, and interviews with maintenance personnel. While several sites above the area of the early construction disturbances were identified, it was learned that the major rockfall hazard occurred due to small particles rolling out of the previously cut uphill scars.

As described earlier, much of the alignment design for the interstate highway focused on “healing” the uphill scars, which both mitigated rockfall and resolved the aesthetic issues.

Nonetheless, the interstate highway design did require new uphill cuts. Besides being aesthetically pleasing, the new rock cuts needed to be safe and stable. As the scope and size of rock cuts was being determined as part of the alignment fine-tuning process, project personnel embarked upon a detailed analysis of rockfall mitigation.

While historic records of rockfall in Glenwood Canyon were being developed, basic information about the design of rockfall protection was gathered. Conventional design of rockfall protection using ditch design criteria was often not applicable for the natural slopes. Construction of wide ditches was not physically possible due to both the topographic constraints and not meeting the aesthetic criteria for the project. A reasonable method of estimating parameters such as probable bounce height and energy of rockfalls was necessary input to the design of innovative elements such as protective fences. This data would also allow more optimum, non-conservative design of rockfall catch ditches.

In 1985, with the availability of personal computers that could be located in a field office, it was decided to develop a rockfall simulation program. The resulting Colorado Rockfall Simulation Program (CRSP) used easily identified input parameters such as slope geometry, slope material properties, and rock geometry and material properties. Using a stochastic model and conservation of energy methods, CRSP provides estimates of probable rockfall bounce heights, velocities and energies for rockfall on natural or excavated slopes. The experimental verification and calibration of CRSP was conducted in conjunction with testing of proposed Glenwood Canyon rockfall fences at a site near Rifle, Colorado. The reader is referred to Pfeiffer and Bowen, 1989 for further information.

With the availability of CRSP, and a demand for innovative rockfall mitigation techniques, work continued on the development of protective fences. As reported by Andrew (1992) and Barrett and White, (1991), several fences and barriers were developed to absorb the substantial energy of rockfall. Fence/barrier selection was based on particular site requirements. For example, the "chime" rockfall attenuator, which is made of steel rods threaded with stacked tires (still on their wheels) and strung on a wire rope suspended across a narrow chute was very appropriate for some locations.

Another benefit of CRSP was the ability to model different rock cut slope designs to verify their safety. With the strong aesthetic requirements to create natural appearing rock cuts, certain features such as benching and planting pockets needed to be incorporated in the design. Those sculpted features would be included only with the proper technical analysis to ensure that they did not create additional rockfall hazards.

The design and construction of rock cuts on I-70 through Glenwood Canyon represents the most visible product of the engineering geologists and geotechnical engineers. While every effort was made by the design team to eliminate rock cuts, in some locations it was physically impossible to fit the 4-lane platform without some rock excavation. When the alignment fine-tuning process progressed to a point that cross-sectional analysis indicated a need for rock excavation, a sub-team of landscape architects, engineering geologists, and surveyors began their work in earnest. Basic questions such as the extent and type of overburden, and structural geology were gathered for the site. Engineering geologists worked hand in hand with landscape architects to transition cut slopes back into the natural terrain, and incorporate sculpted features for a natural appearance. It became evident early in the project that existing seams and natural fracture features could be used to shape the horizontal and vertical sculpted features. By using these natural structural characteristics of the rock formation, safe, stable, and natural appearing cuts could be created.

After field staking and additional cross-sectional analysis was completed, a preliminary cut slope design, with transitions to existing ground, was developed. This design was then analyzed using CRSP to determine whether protective structures, or modified ditch designs would be necessary. Final adjustments were made to address all safety, geologic, geotechnical, constructability and aesthetic considerations, and final roadway cross-sections were drawn.
Plan sheets included a sheet of typical rock sculpting concepts, so the contractor had knowledge of what needed to be accomplished. In addition, the contractor had to submit construction access plans, blasting patterns, and muck removal plans for the engineer's approval prior to beginning the cut. The costs for this work was included in the per cubic yard price for unclassified excavation (special). Force account was only paid for additional shaping and grading directed by the Engineer during excavation. It was a significant challenge to field personnel (both on the contractor side and CDOT side) to plan, manage, and execute earthwork operations to minimize the amount of extra force account work. Not only was this an issue of cost control, but it also had the potential to impact the critical path schedule, as the rock excavation was usually an early work item to be accomplished before structure construction could begin.

Rock blasting crews had to take a different approach to the blast designs in Glenwood Canyon. Because of traffic handling and the proximity of historic structures (D&RGW railroad facilities, Shoshone Hydroelectric Plant), pre-blast surveys were taken on these structures, and stringent specifications were included for blast effect control measures. Small shots were necessary to meet the above requirements, to minimize damage to intact adjacent rock, and to allow the expeditious handling of blasted rock with the strict traffic control requirement. Powder factors ranged from about a maximum of 3.7 pounds/cubic yard for tunnel blasting (Harrison and Revey, 1991) to 0.2 – 0.8 pounds/cubic yard for surface blasting.

Local stability of rock cuts was a primary concern of the design/construction team. As cuts were taken down, the structural geology of the as-excavated condition was carefully evaluated for stability. Based on these results, and the expert judgment of the engineering geologists, rock reinforcement was often required. Fully encapsulated rock bolts, with epoxy-coated bars and hardware, were used. Much of the bolting was done from a drill rig mounted on a man-basket, lifted by a crane (Figure 15). Safety and stability of the final cut was a primary concern.

After blasting and stabilization, the freshly cut rock face was washed down, and rock stain was applied. Several different staining methods were used. Early in the project, a clear substance was applied, which darkened due to a photochemical effect. Where darkening the rock was not appropriate, actual paints were used at some locations. The effort of rock sculpting and staining was very successful, as evidenced by a former Governor of Colorado congratulating the design/construction team on the wonderful "fit" of the French Creek viaduct to what he perceived to be the natural formation. In fact, over 250,000 cubic yards of rock were removed to create a roadway bench in this area (Figure 16).

As rock excavation progressed on the project (both for surface cuts and tunnel excavation), innovative strategies for muck handling and storage were used. Very little rocky material was wasted, as evidenced by the small waste pile that exists today next to I-70 at the Dotsero interchange. Generally, larger particles of muck were used for riprap. Other rock was processed through grizzlies, and used for embankment behind walls. The majority of the tunnel muck was used as embankment behind the west bound retaining wall between the Shoshone Dam and power plant. The remainder was used as fill at the site of the No Name Rest Area, to replace river gravels borrowed from that location on an early construction project.

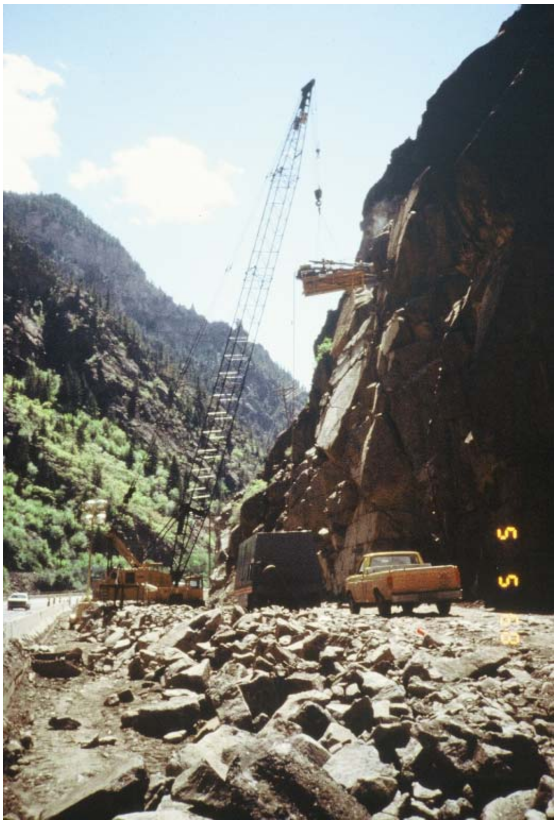

Figure 15. Rock reinforcement installation.

CONCLUSION

The successes of Interstate 70 through Glenwood Canyon are recognized through the many awards the project has received. These awards include the 1993 ASCE Outstanding Civil Engineering Achievement Award (DMJM, 1993) and the 1994 Outstanding Environmental and Engineering Geologic Project Award from the Association of Engineering Geologists.

The success of the project serves as an outstanding model for a multi-disciplinary team approach in complex projects on difficult terrain. By innovation through teamwork, this $500 million

Figure 16. Rock excavation and bridge construction, French Creek

project demonstrates the value of a strong role for the engineering geologists and geotechnical engineers on multi-disciplinary teams. As reported by National Geographic Society, (1992), "Engineering is essentially a social art, increasing in importance as life becomes more complicated. We depend on it as we do on nature itself, for it makes nature more tractable and accessible."

ACKNOWLEDGEMENTS

The authors would like to acknowledge the individual contributions of Mr. Shan-Tai Yeh, P.E. to the success of the Glenwood Canyon I-70 project. Mr. Yeh was responsible for the geotechnical design of nearly every major foundation element on the project. Messrs. John (Brandy) Gilmore, Robert (Bob) Barrett, and Nelson Chou had significant roles and responsibilities in the decision-making on geologic issues. Messrs. Tim Bowen and Paul Macklin were amongst the engineering geologists and geotechnical engineers who made the right things happen in the field. The final appearance of the project was envisioned by Mr. Joseph Passonneau (No Name to Hanging Lake), and the late Mr. Edgardo Contini (Hanging Lake to Siloam Springs). It was these "inspired designers" who formulated the vision of the terraced alignment, tunnels and viaducts.

The successful geotechnical engineering of the Glenwood Canyon I-70 tunnels is due to the skilled expertise of Messrs. Tom Lang (deceased), John Bischoff, Bill Hansmire, Randy Essex, Anthony Caserta, Dan Louis, Steve Klein and Tom Scotese (deceased).

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DIFFICULT GEOLOGICAL CONDITIONS ON STEEP SLOPES THAT REQUIRED INNOVATIVE SLOPE STABILITY AND RETAINING WALL SYSTEMS FOR HIGHWAY WIDENING AT SNOWMASS CANYON

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Key Terms: slope stability, collapsible soils, hydro-compaction, retaining wall systems, soil nails, ground anchors, micropiles

ABSTRACT

Yeh and Associates has been involved with the Colorado Department of Transportation in the evaluation, analysis and design for improvement of four miles of Snowmass Canyon - State Highway 82, near Aspen, Colorado. This section of highway is located in a complex geologic area with slopes in excess of 45^0. Yeh and Associates role on the project was to provide a geologic / geotechnical investigation, slope stability evaluation, and retaining wall design.

The geologic and geotechnical investigation utilized helicopter transported drilling rigs. The subsurface conditions were evaluated by drilling 289 borings with this equipment. Geologic hazards that were identified on the project included landslides, rockfall, and collapsible soils. The competency of some of the bedrock in the area was affected by faulting.

The roadway alignment was designed with both cut and fill retaining walls. Global stability was typically the controlling factor for the design of the wall systems. The retaining walls for the project consisted of soil nail walls, mechanically stabilized earth (MSE) walls, ground anchor / tieback walls, and micropile foundation walls. Because of the steep terrain, MSE wall sections were over 40 ft (12 m) in height and ground anchor and soil nail wall sections were over 25 ft (3.6 m) in height.

INTRODUCTION

State Highway 82 in Colorado is the primary two-lane route between Aspen and Glenwood Springs. The highway follows along the general path of the Roaring Fork River that flows from Aspen down to the confluence with the Colorado River. The $93 million Colorado Department of Transportation (CDOT) project was undertaken to expand a 3.4-mi (5.4 km) section of the existing state highway from two lanes to four lanes, creating two lanes up valley and two lanes down valley. The up valley lanes are located on the slopes above the existing two-lane highway. The down valley lanes follow the general alignment of the existing highway. Figure 1 illustrates the general layout of the project area.

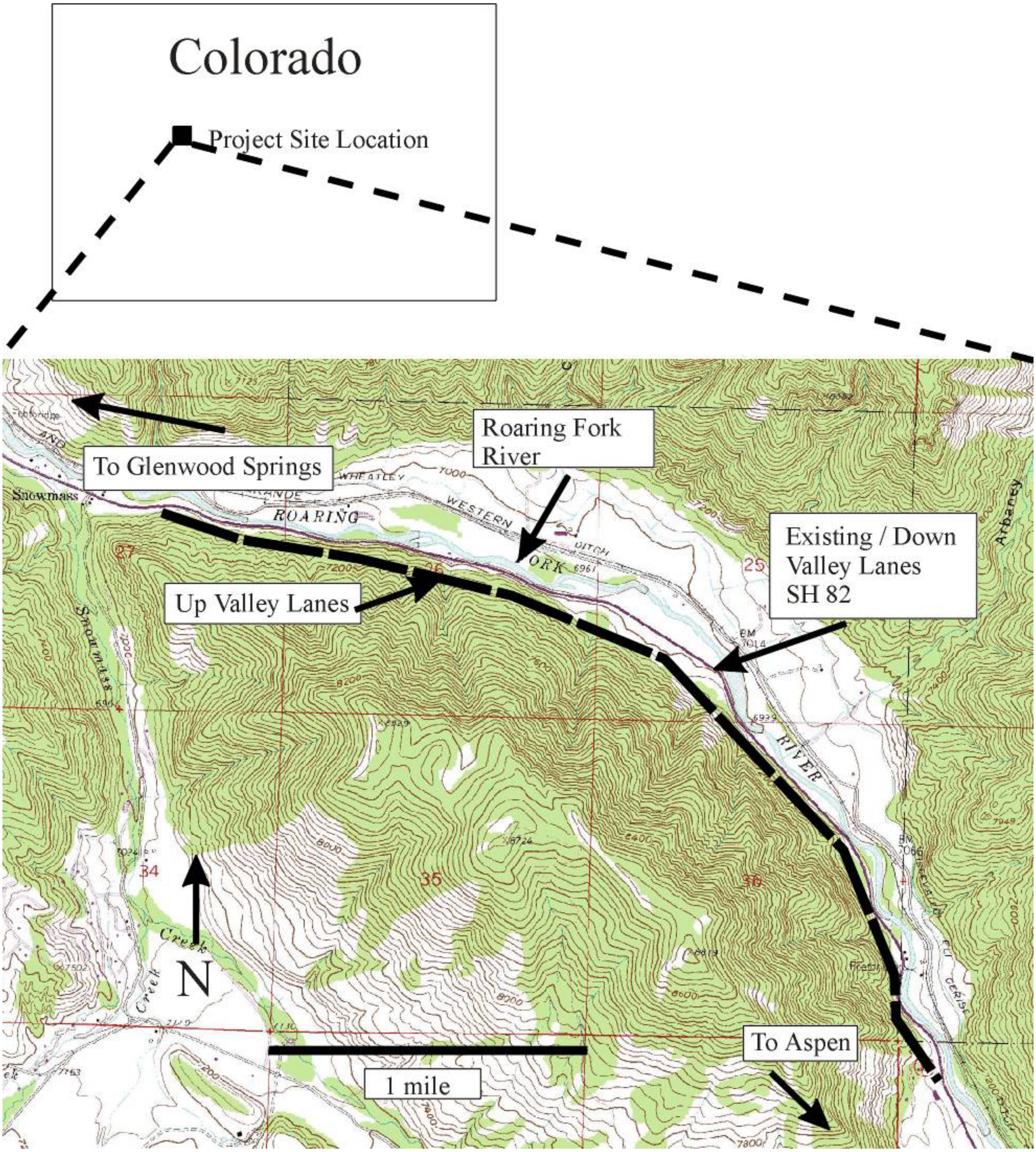

Figure 1. General layout of the project area.

The expanded roadway section located in Snowmass Canyon is characterized by steep terrain and variable geologic conditions. In many areas the slopes are in excess of 45^0 and hydro-collapsible subsurface materials are located throughout the project. This paper addresses the geologic and geotechnical aspects of the project that controlled the wall design in the canyon. The paper also describes mechanically stabilized earth (MSE) walls, soil nail walls, ground anchor (tieback) walls, micropile foundation systems, and rockfall mitigation utilized on the project.

SITE GEOLOGY

The underlying geology of the project site consists of steep colluvial slopes, debris fans, alluvial stream deposits and moderately dipping sandstone bedrock. Major and minor faulting is also evident within the project area. The colluvial materials consist of angular, poorly sorted rock debris embedded in a matrix of silt, sand and clay. Most of the colluvial material is present on steep slopes that are in excess of 1H : 1V.

A majority of the steep slopes in the project area are comprised of debris fan depositional features resulting from episodic debris flows that formed after intense storm events. Relatively large natural catchment basins located above the highway act as source areas for the debris flow materials. Thin layers of charcoal deposits are exposed in many of the debris fans that have been cut by the existing alignment. The charcoal remnants are an indication of past fires, which burned the upper slope vegetation and led to mass wasting of surface materials during storm or runoff events. The debris fan materials were typically loosely placed in many areas particularly along the boundaries of the mass wasting events creating weak, uncompacted material. Figure 2 depicts a typical debris fan road cut profile exposed along the existing highway.

The debris fan deposits can exhibit the potential for collapse when wetted and can be considered hydro-compactive in some areas. The term hydro-compactive soil refers to subsurface deposits that have the potential to decrease in volume by the addition of water. Typically this decrease in volume is independent of any changes in the vertical loading of the material. The process of soil collapse by inundation with water has been described in a variety of ways and has been referred to as hydro-compaction, hydro-consolidation, collapse, settlement, shallow subsidence, and near-surface subsidence. Additional subsidence can be caused by dissolution of disseminated sulfates or other soluble materials within the soil that dissolve when saturated with water.

Alluvial stream deposits, which were deposited in the past by the Roaring Fork River, can be found on elevated terraces on the slopes of Snowmass Canyon and along the banks of the present day river. The alluvial deposits can range in thickness from a few feet to as much as 40 ft (12 m). The deposits generally consist of sandy, well-rounded gravels, cobbles and boulders that are well graded or poorly sorted. These materials are highly desirable for foundation materials since they have high strength and bearing capacity and are not susceptible to settlement or collapse.

The bedrock in the project area is predominately comprised of the Maroon Formation with overlying State Bridge Formation in the up valley section. The Maroon Formation in this area is more than 2500 ft (760 m) thick and consists of reddish sandstones and siltstones. The State

Bridge Formation is similar to the Maroon, consisting primarily of hard reddish brown sandstone with some siltstone. The bedrock is generally competent, ranging from hard to very hard, except where fractured by faulting. The Rock Quality Designation (RQD) generally ranges from 50 to 90 percent in the unfaulted or unfractured areas. In areas were faulting has occurred the RQD is 0 to 10 percent. Strikes and dips in the bedrock vary throughout the project. In the up valley areas most of the bedrock is relatively uniform with an approximate 25 to 35 degree dip. Figure 3 shows an area with colluvial, alluvial and bedrock materials.

Figure 2. Debris Fan Deposits.

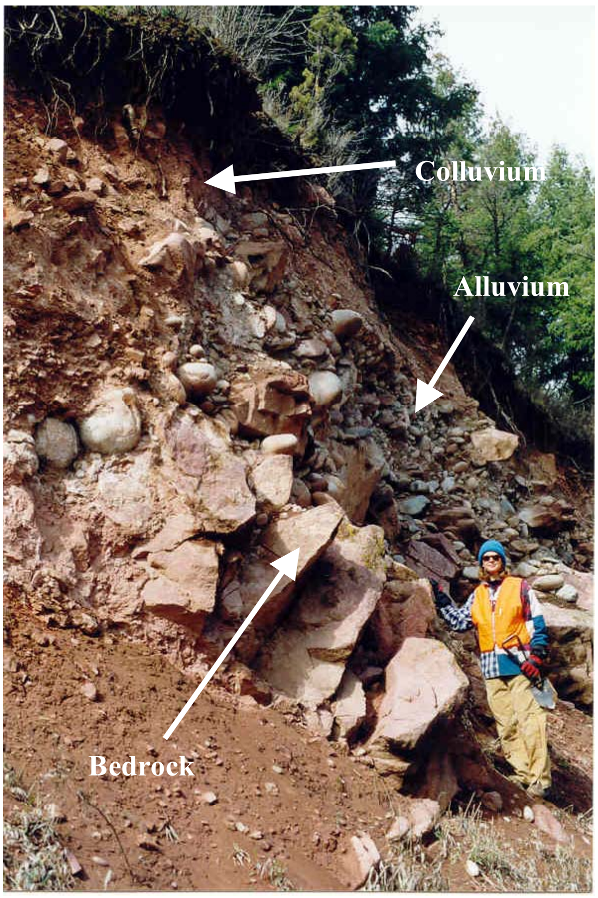

Figure 3. Colluvial, alluvial and bedrock outcrops.

Generally the site is relatively dry with groundwater levels only being observed near the elevation of the river. Large storm events with greater than normal precipitation tend to produce debris flows in the less vegetated sections of the project.

GEOLOGICAL / GEOTECHNICAL INVESTIGATION

To characterize the geologic conditions and subsequent constraints to construction it was necessary to implement a geological/geotechnical investigation. The investigation included surface mapping, identification of geologic hazards, and subsurface drilling. Surface mapping was undertaken to identify potential debris fan areas, collapsible soil areas, and identification of subsurface materials along the road cuts. Areas that had the potential for landslides and rockfall were also identified.

Due to the steep slopes encountered on the project it was necessary to use helicopter transported coring drill rigs to access most of the sites. A total of 289 borings were completed over a two-year period. The most extensive drilling period utilized five drill rigs and one helicopter. Split spoon samples were obtained for subsurface materials above the bedrock. Core samples of the bedrock were obtained to evaluate rock quality. Figure 4 shows one of the drill rigs operating in a steep section near the Roaring Fork River.

Results from the geotechnical investigation identified areas of potential soil collapse from hydro-compaction, location of alluvial and bedrock elevations, and general subsurface material characteristics. Additionally, a bedrock elevation map was generated from the drill hole information and used in the profile sections for the proposed highway alignments.

Figure 4. Drilling on the side of Roaring Fork River.

SLOPE STABILITY AND DESIGN OF RETAINING WALL SYSTEMS

The two predominating design constraints for the project were the relatively steep slopes in excess of 1H : 1V for most of the up valley lanes, and the potential for encountering hydro-compactive or collapsible soils throughout the area.

Prior to the retaining wall design phase of the project, it was necessary to evaluate the existing global stability of the area. Soil strength parameters were determined from sample testing of the subsurface materials and from back-calculation of the existing slopes. It was assumed most of the slopes throughout the project were at or above an existing factor of safety of one. Some sections of the project exhibited minor circular failures and sloughing of the colluvial materials, however no large-scale circular failures were observed within the project area prior to construction.

After evaluation of the existing conditions at the site, it was necessary to evaluate and model the slope stability of more than 31 critical sections along the proposed up valley and down valley alignments. Slope stability analysis considered not only the final completed alignment profile, but also the temporary conditions that were anticipated to occur during the construction sequence. Based on this analysis, it was necessary to construct temporary berms along many sections of the alignment to maintain an appropriate slope factor of safety during the construction phase of the project. For the proposed highway alignment a static slope factor of safety of 1.3 and a seismic slope factor of safety of 1.01 was used for the retaining wall systems.

The existence of collapsible soils was also a general constraint to the design that required foundations of retaining wall structures to be located on alluvial materials, bedrock, or re-compacted structural backfill with geogrid reinforcement. The geotechnical data obtained from the drilling program was a valuable tool for delineating the elevations of the colluvial, alluvial and bedrock materials.

Based on the characterization of the geologic and surface conditions at the site it was determined that Mechanically Stabilized Earth (MSE) Walls with soil nails or ground anchor/tieback excavation support was a viable retaining wall system for the proposed highway alignment. Figure 5 depicts a generalized analysis of a slope stability model with a MSE wall and ground anchor tieback system.

MECHANICALLY STABILIZED EARTH WALLS

Mechanically Stabilized Earth Walls (MSE) were used on the site since they typically tolerate greater settlement and deflection than traditional rigid retaining wall designs. This was especially important since the potential for collapsible soils was present. Where feasible on the project, the base of the MSE walls were located on bedrock or alluvial materials, however, certain colluvial sections of the project site were too thick for reasonable excavation. In these areas subexcavation was necessary. Sub-excavation consisted of removing a vertical portion of the colluvial material and replacing it with structural backfill. In higher wall sections or areas thought to be more prone to collapse, colluvial materials were excavated and replaced with a geogrid "sub-wall" foundation. The "sub-wall" was designed to act as a foundation mattress for the MSE wall.

The MSE walls were constructed using pre-cast concrete panels with a geogrid reinforcement system. The panels were shipped to the project site and positioned with temporary support on the front face of the walls. MSE walls on the project ranged from 5 ft to 41 ft (1.5 m to 12.5 m) in height for an approximate total of 315,000 ft^2 (30,000 m^2) of wall face. Figure 6 depicts a generalized profile of the MSE wall system for the project. Figure 7 shows the back of the MSE wall during fill placement.

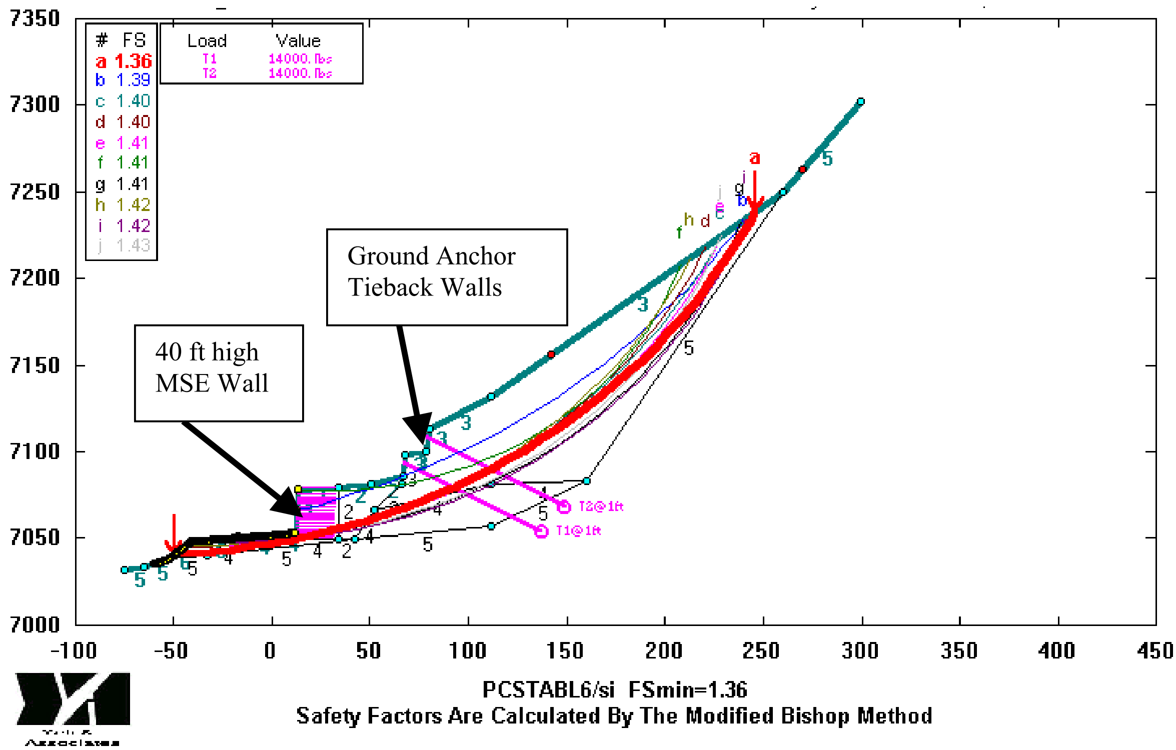

Figure 5. Generalized slope stability analysis depicting an MSE Wall with Ground Anchor Tiebacks for global stability.

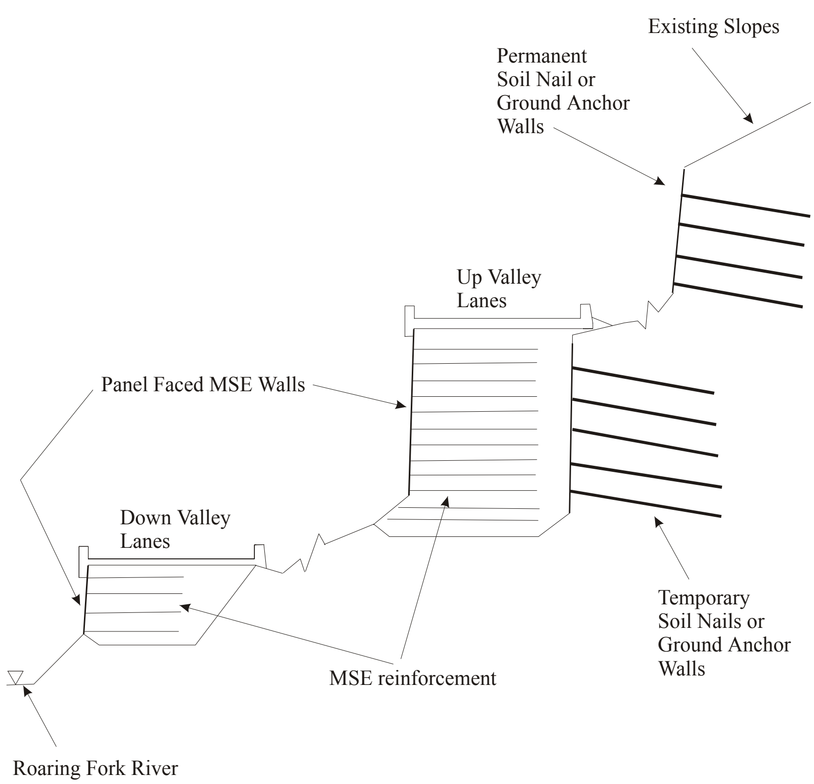

Figure 6. Generalized Profile of Retaining Wall System for the Project.

Figure 7. Fill Placement for MSE Wall.

SOIL NAIL WALLS

Soil nail walls were used for excavation and global stability support for a majority of the cuts along the proposed alignment. Both temporary and permanent soil nail walls were used. Final facing of the permanent soil nail walls consisted of pre-cast panels. The soil nail wall heights for this project ranged from 3 ft to 25 ft (0.9 m to 7.6 m) high with reinforcement nail lengths ranging from 15 ft to 30 ft (4.5 m to 9 m). A 4 ft (V) by 5 ft (H) (1.2 m x 1.5 m) spacing of the reinforcement soil nail bars was used.

The soil nail wall is constructed in a top-down sequence. An initial 4 ft (1.2 m) vertical cut was made in the slope. If the cut was not self-supporting, a temporary berm had to be used. Soil nails, which consisted of all thread bar, were placed into predrilled holes on a designed pattern. The holes were grouted either prior to bar placement or by tremie methods. Welded wire fabric with drainboards, to insure groundwater drainage of the system, were then placed over the nails and exposed cut face. The soil nails were then connected together by horizontal and vertical waler reinforcement bars with plates attached to the ends of the nails. Shotcrete was then placed to develop a complete structural reinforcement system. The sequence continued in a top down manner so that no more than 5 ft (1.5 m) of vertical excavation was exposed during construction. As earth pressures develop behind the shotcrete facing system, the load is transferred through the facing system to the soil nail bars. The soil nail bar then transfers the load into the bonded section of the nail. Small deflections at the top of the walls are generally anticipated and designed for. A final precast panel is then attached over the wall face. Approximately 56,600 ft^2 (5,200 m^2) of permanent soil nail walls were constructed on this project.

Figure 8 shows the installation sequence for the second row of a soil nail wall. Soil nails and shotcrete facing have been completed for the top row. Figure 9 shows the placement of precast panels over the completed permanent soil nail walls.

Figure 8. Soil Nail Wall Construction.

Figure 9. Attaching final pre-cast panel facing over soil nail wall.

GROUND ANCHOR WALLS

Many areas along the existing slopes were too steep to support effectively with soil nail walls. In these areas, it was necessary to use ground anchor (tieback) support. The ground anchors consisted of multiple strand tendons that ranged in length from 35 ft to 70 ft (10 m to 20 m). Bond lengths of the ground anchors averaged 20 ft (6 m). Due to the overall low bond strength of the colluvial materials it was necessary to place the bond lengths of the anchor systems in bedrock or alluvial materials. The ground anchors were set on an 8 ft horizontal by 8 ft vertical spacing (2.4 m x 2.4 m). One to three rows of permanent tiebacks were used to provide excavation support and to satisfy global stability where necessary. The ground anchor support panels consisted of 8 ft by 8 ft (2.4 m x 2.4 m) rebar reinforced sections that were shotcreted in place. Figure 10 shows installed ground anchors. Workers are putting a grout containment device on an anchor that is to be installed. A coiled ground anchor is visible next to the workers.

The ground anchor walls were constructed in a top-down manner similar to the soil nail walls. Approximately 35,000 ft^2 (3200 m^2) of permanent ground anchor tiebacks were used on this project. Figure 11 shows two completed rows of a ground anchor / tieback wall.

Figure 10. Ground Anchor Installation.

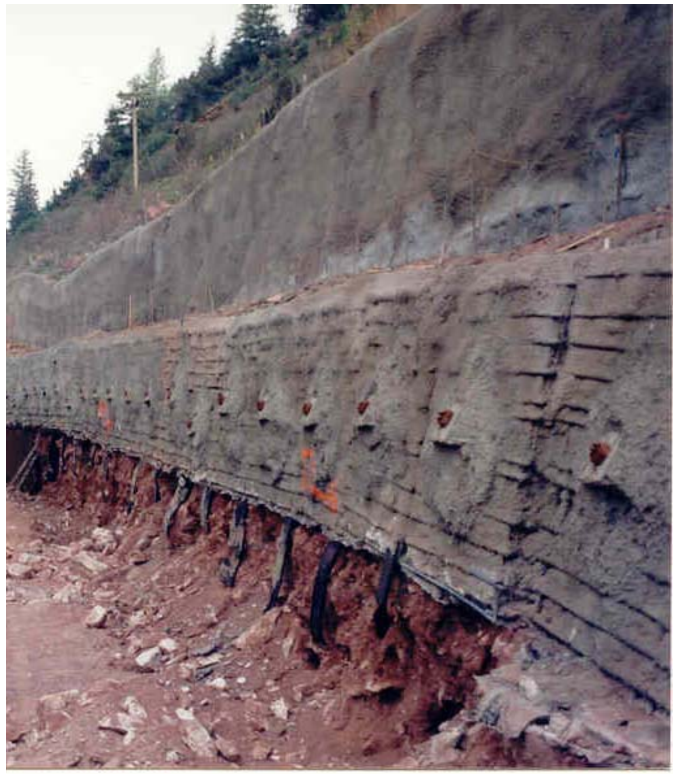

Figure 11. Two rows of a ground anchor wall.

MICROPILE FOUNDATION SUPPORT

In certain sections of the project where the side slopes were in excess of 1H:1V and the bedrock quality was relatively low, double tee retaining walls with micropile foundation support were used. These walls ranged in height from 14 ft to 36 ft (4.2 m to 11 m) for approximately 1700 ft (520 m) of wall length. The original foundation design called for 30-in (76 cm) diameter caisson (drilled shaft) support of the wall system with a 10 ft to 12 ft (3.0 m to 3.7 m) spacing between the caissons. The foundation design was modified to use smaller diameter micropiles with variable spacing from 2-½ ft to 8-½ ft (0.76 m to 2.6 m) depending on the wall height. For higher wall heights it was necessary to use a tighter spacing.

The micropiles consisted of an approximate 7 in (18 cm) casing that was drilled through overburden materials and into bedrock a minimum of 2 ft (0.6 m). A "rock socket" was then drilled further into the bedrock to form the bond zone of the micropile. An all-threaded bar was placed inside the casing and extended into the "rock-socket". The inside of the casing and the rock socket were then grouted. Plates were attached to the top of the threaded bar and the system was incorporated into the poured foundation for the double tee walls. To provide additional external wall and global stability it was necessary to incorporate permanent ground anchors into the foundation system. Ground anchors were placed in-between the uphill row of micropiles.

Figure 12 illustrates the generalized profile of the double tee wall system supported by micropiles and ground anchor tiebacks. Figure 13 shows the footing prior to placement of concrete. Note the square plates attached to the top of each micropile. The ground anchor tiebacks are placed through the pipes, which are inclined in the left center area of the figure.

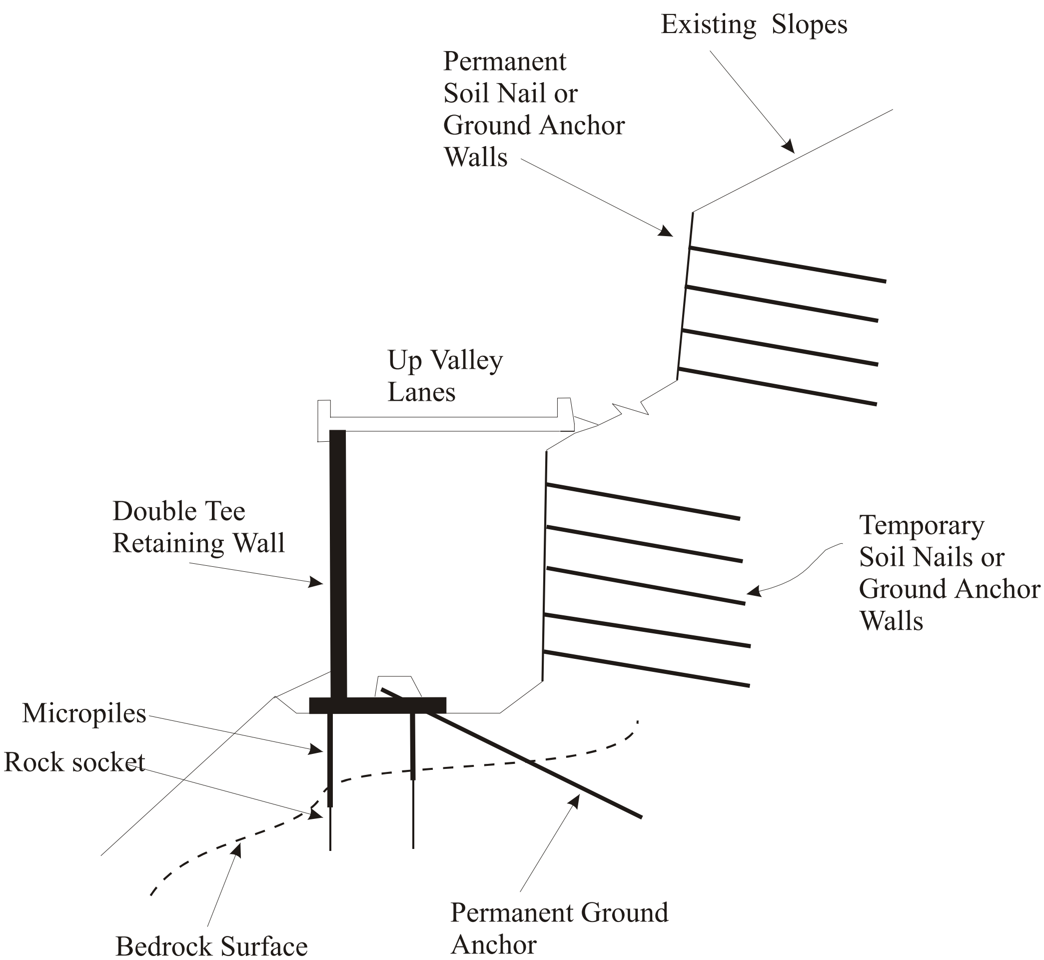

Figure 12. Generalized Profile of Retaining Wall with Micropile Foundation.

ROCK BOLTING AND DRAPED MESH

Most of the slopes above the highway alignment consist of colluvial materials, however certain sections of the alignment are next to steep bedrock cliffs. In these areas, kinematic slope stability analyses and the Colorado Rockfall Simulation Program (CRSP) were used to recommend appropriate mitigative options for rockfall. Typically, rockfall mitigation consisted of spot bolting, pattern bolting, and the use of draped mesh in critical sections. Figure 14 shows the use of a crane basket for bolting a rock cut section that will be located adjacent to a bridge structure. A minimum of 10,000 ft^2 (929 m^2) of exposed rock-cuts will have been meshed by the completion of the project.

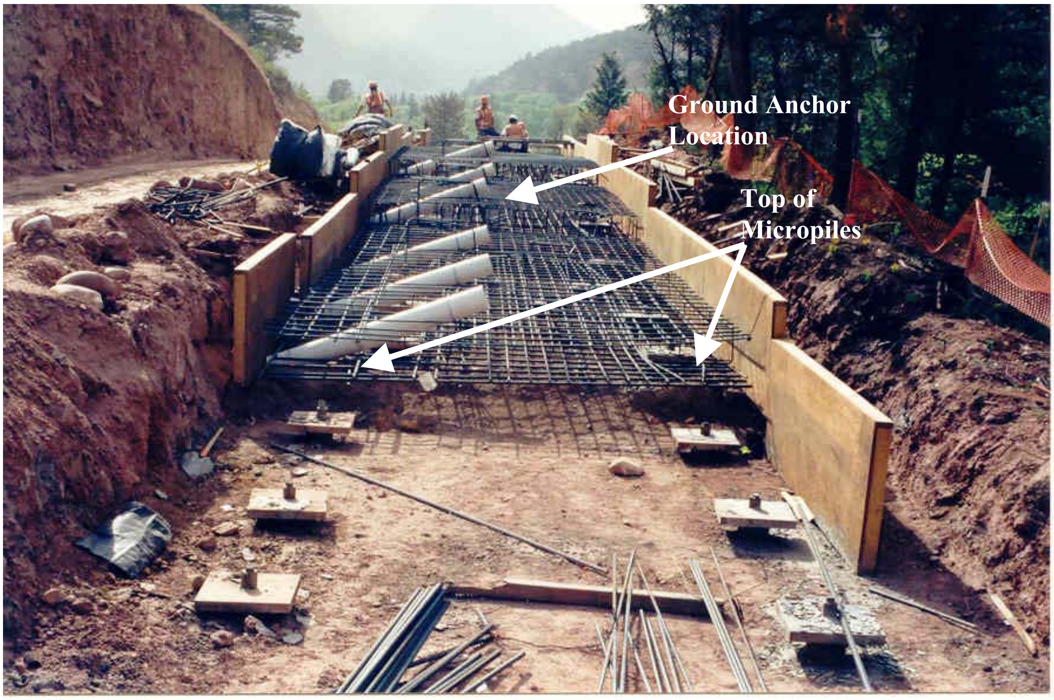

Figure 13. Double tee footing foundation prior to concrete placement.

Figure 14. Rock bolting from crane basket.

SUMMARY

The final completion of the project is expected to occur in the fall of 2005. Overall, the wall systems proposed were based on the subsurface information obtained from the geotechnical investigation. The use of helicopter transportation of the drill rigs was an invaluable resource for obtaining information in many locations that were not accessible until actual construction commenced. Without the extensive geotechnical investigation, it would have been difficult to predict and design for many of the conditions that are present at the site.

ROCKFALL POTENTIAL ALONG I-70 AT THE GEORGETOWN INCLINE

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Key Terms: rockfall, rockfall rating system, Colorado rockfall simulation program, modified Q system, weathering processes, rockfall fences

ABSTRACT

This report presents the results of the evaluation of the rockfall potential along the Georgetown Incline for the Colorado Department of Transportation (CDOT). This report was conducted to review available CDOT information on rockfall events, review the mechanisms and conditions that contribute to rockfall, evaluate the potential sources for rockfall using the modified Q method, and analyze rockfall from selected areas of the project site using the Colorado Rockfall Simulation Program. It should be noted that rockfall events are sporadic and unpredictable. This report attempts to comment on the rockfall potential along the Georgetown Incline, but does not attempt to predict the recurrence interval, magnitude, or location of a rockfall. These factors cannot be predicted. Consequently, neither the rockfall hazard in terms of probability of a rockfall at any specific location, nor the risk to people or facilities from such events, are assessed in this report. Furthermore, along the Georgetown Incline rockfall can potentially occur at any time and at any location.

OVERVIEW

The Georgetown Incline consists of a 2.2 mi (3.5 km) section of Interstate 70 (I-70) between Georgetown and Silver Plume, Colorado. Figure 1 is a topographical map illustrating the general site area. Figure 2 illustrates the vertical relief of the site looking to the west. The study area is located between approximate highway mile markers (MM) 225.7 to 227.9 on I-70. In this section, the overall elevation change of the site is approximately 500 ft (150 m), with westbound I-70 climbing uphill from Georgetown to Silver Plume at grades ranging from 5 to 8 percent, and eastbound descending from Silver Plume to Georgetown at similar grades. The highway alignment is cut into relatively steep mountainous slopes that exceed 1H:1V in many places with numerous exposed bedrock outcrops located above the highway. The natural backslopes above the current highway alignment exceed 1,700 ft (520 m) vertically throughout much of the project area. Cutslopes and disturbed areas just above the highway generally range from 20 to 150 ft (6 to 45 m) vertically with cutslope angles ranging from vertical to 60 degrees. The surficial materials comprising the steep backslopes above the current alignment typically consist of colluvial, talus, and isolated mine tailings deposits. Numerous bedrock outcrops form vertical cliff faces in many locations creating potential source areas for rockfall. The combination of steep slopes, relatively loose surficial materials and particle sizes ranging from silt to boulders, creates an area that has experienced numerous rockfall events in the past.

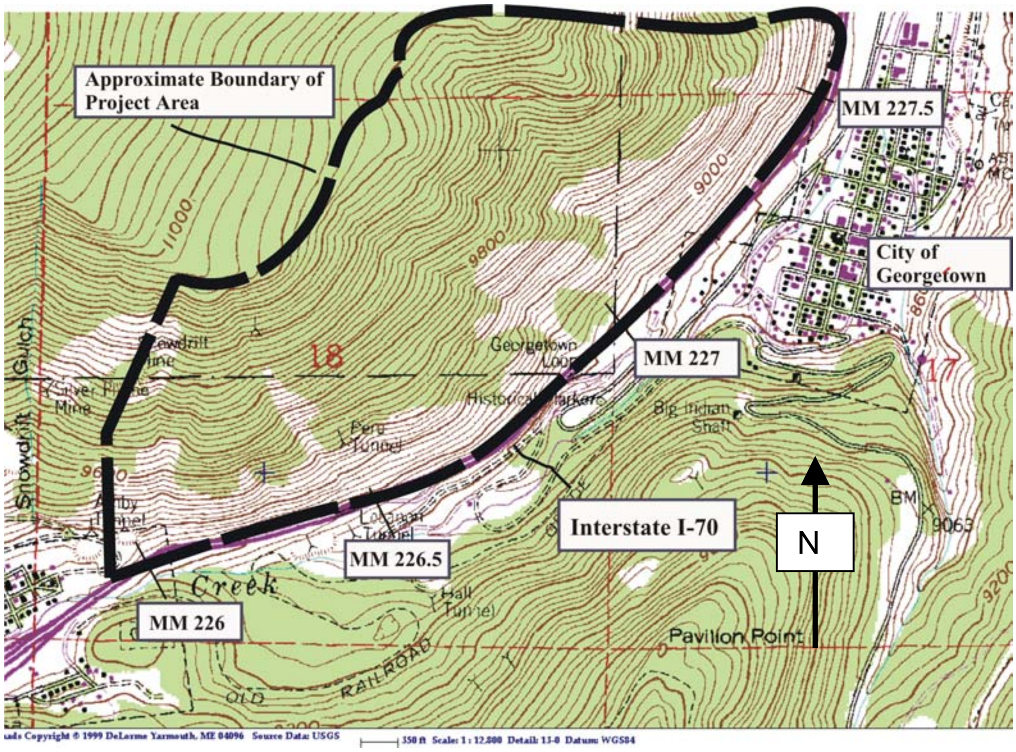


Figure 1. Georgetown Incline topographic map.

Figure 2. Georgetown Incline project area, view looking to the west.

SITE GEOLOGY

The Georgetown Incline is located in a U-shaped valley that was likely glaciated up until 12,000 to 14,000 years ago (Madole, 1998). Scour and removal of overburden material resulting from glacial activity created oversteepened slopes that periodically experience rockfall. Additionally, the steep slopes above the current highway alignment typically consist of colluvial and talus surficial materials that can be eroded by surface water runoff and/or wind action which contribute to rockfall events.

The bedrock in the area consists primarily of the Silver Plume Granite, a pink to pinkish-gray granite that consists primarily of microcline, plagioclase, and quartz, with minor biotite and muscovite. This granite has been age dated at approximately 1,400 million years. Other bedrock in the area consists of metamorphic gneiss that has been described (Widmann and Miersemann, 2001) as a sillimanitic biotite gneiss, interlayered felsic and hornblende gneiss, and an interlayered felsic, hornblende, biotite and calc-silicate gneiss. Early proterozoic migmatites have also been mapped. The migmatites are described as rocks that have been heavily intruded by granitic material and/or have been intensely deformed and heated to the point of partial melting.

Underground mining was also prevalent along the Georgetown Incline and likely left stopes and other underground openings within the hillside northeast of Silver Plume. Mine adits (tunnels) are also visible at numerous locations along the Georgetown Incline increasing in density near the town of Silver Plume. Groundwater drains from an adit year round near the overlook at approximate Mile Marker 226.7. It is likely the lowest adits or tunnels effectively drain most of the subsurface water at the Georgetown Incline.

PROCESSES THAT CONTRIBUTE TO ROCKFALL

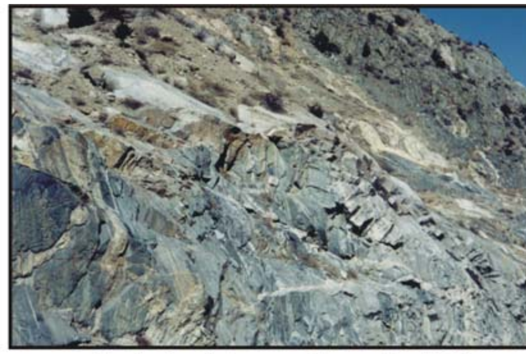
Rockfall originates in both rock and soil. Rockfall as defined by Cruden & Varnes, 1994: "The detachment of soil or rock from a steep slope followed by movement of the material by falling, saltation, or rolling. Movement is very rapid to extremely rapid. Except when the displaced mass has been undercut, falling may be preceded by small sliding or toppling movements that separate the displaced material from the undisturbed mass". This definition describes the actual rockfall event, however there are a multitude of conditions and mechanisms that lead up to this event. In general, the typical short-term conditions that determine if a rockfall event is to occur can be related to the rock mass characteristics, precipitation history (which includes freeze and thaw action), bio-disturbance, and the effects of wind scour. Long-term processes that contribute to rockfall include erosion of the surface materials related to mountain building events (orogenies), glacial processes, climatic changes, exfoliation, and chemical weathering of the rock mass.

Rock Mass Characteristics

The stability of rock slopes is typically dependent on the characteristics of the discontinuities of the rock mass. Terzaghi suggested that the critical height of slopes with unweathered rock without any discontinuities would be roughly 4,200 ft (1280 m) (Terzaghi, 1962). This of course is typically not the case since rock masses have varying types of discontinuities. The rock discontinuities are typically described in terms of orientation, continuity, infilling, aperture, spacing, and roughness. The orientation and characteristics of the discontinuities in the rock mass, relative to the face of the slope, effect slope stability and the type of failure that potentially can occur. Analysis methods to determine rock slope stability based on discontinuity orientation have been described in detail, the most notable publication by Hoek and Bray (1981).

Based on an evaluation of the discontinuities and relevant groundwater conditions observed at the surface, it is usually possible to develop a relative rock mass rating for a given bedrock outcrop. Figure 3 depicts photos of various bedrock outcrops in the project area that illustrate typical bedrock conditions and discontinuities.

Typical bedrock outcrops above cutslopes



Typical cutslopes adjacent to I-70

Typical bedrock outcrops above cutslopes

Typical colluval and alluvial slopes above cutslopes

Figure 3. Typical bedrock outcrops along the Georgetown Incline.

Precipitation

Precipitation and snowfall with associated freeze-thaw activity appear to be the most significant triggering mechanism of rockfall on the Georgetown Incline. Figure 4 suggests a correlation between an increase in reported accidents and increased precipitation when the average temperature is near the freezing point typically during the spring months of March, April, and May. As discussed previously it appears that freeze-thaw cycles likely contribute to a

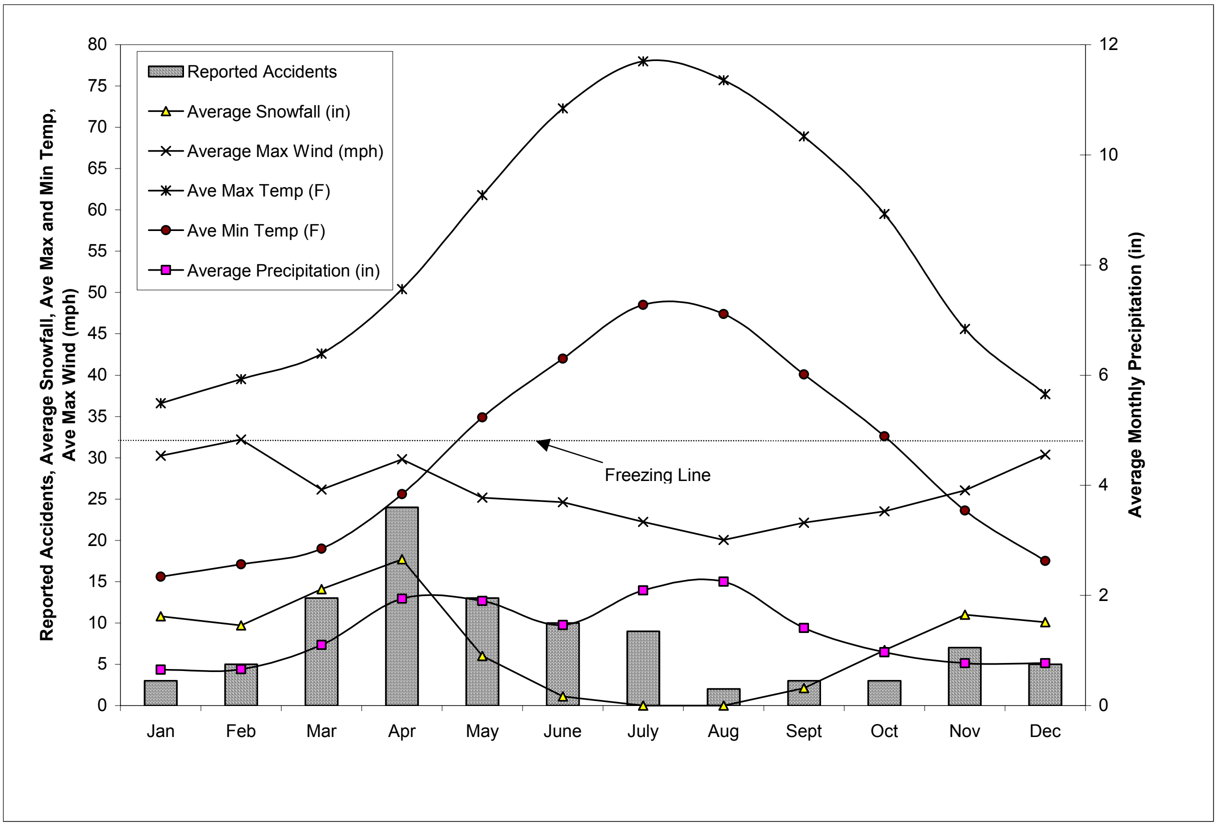


Figure 4. Monthly average snowfall, precipitation, and temperature between 1948 to 1979 and 1995 to 2000 with CDOT accident information from 1976 to 1999. (Sources: Western Regional Climate Center, William Wilson, CDOT)

greater rockfall potential. This correlation between increased freeze-thaw and rockfall appears to be supported by Terzaghi (1962) in his description "Rockfalls involve the intermittent detachment and fall of one or more blocks of rock owing chiefly to the weakening effect of frost wedging and important seasonal temperature changes". Terzaghi based his statement on work done by the Norwegian Geotechnical Institute (Bjerrum and Jorstad, 1957) that suggested a higher rockfall/rockslide frequency in April during increased snowmelt and in October/November when colder temperatures in combination with precipitation are prevalent in Norwegian fjord areas. It was reported that the majority of the rockfalls/rockslides occurred in April since the joint discontinuities were plugged with ice preventing water from draining out of the rock mass. As more water from snowmelt infiltrated into the rock discontinuities, hydrostatic pressures within the fractured rock mass increased eventually causing rockfall. An increase in the hydrostatic pressure can also occur during summer months when surface water infiltration into rock fractures occurs after heavy precipitation events. These triggering events are typically not as common since water can drain out from the fractures more easily when they are not frozen. Additionally, the wedging action resulting from the freezing and expansion of water is typically not present.

Periods of heavy precipitation or rapidly melting snow can also erode the fine-grained materials on the surficial slopes causing larger boulders to dislodge and roll. In many cases an intense precipitation event will lead to a debris flow, which can be preceded by a rockfall event.

Bio-disturbance

Bio-disturbance includes both flora and fauna effects on the slope. The flora or vegetative cover can either increase or decrease the overall rock slope stability and influences rock rollout. Typically, a well-vegetated slope with dense trees will reduce distance that a rock can effectively roll. Trees will also help to reduce the erosional effects of rainfall and rapid snowmelt by reducing runoff. Conversely, a denuded slope due to logging or forest fires can substantially reduce obstacles for a rolling rock, thereby increasing the rollout distance. Denuded slopes will also tend to erode more rapidly. Trees can weaken exposed bedrock surfaces by growing roots into the bedrock discontinuities and effectively jacking the bedrock block loose over long time periods. This increase in joint aperture decreases the frictional resistance along discontinuity surfaces and increases the susceptibility of the rock mass to greater frost penetration. Faunal disturbances on the slopes include construction activities and bighorn sheep. Construction activities are usually limited to periodic rock scaling activities. Bighorn sheep are indigenous to the steep slopes and periodically dislodge material. However, it is not known if they contribute significantly to the rockfall.

Wind

Wind can exacerbate rockfall potential by removing small particles that support larger material and by causing movement in the root zone. Wind may cause high leverage forces on trees, which may dislodge rocks and lead to rockfall. During the spring months at the Georgetown Incline wind velocities average over 25 mph (40 kph).

Weathering Processes

Terzaghi (1962) indicated that the rate of weathering of hillslopes and the nature of the process of weathering depend on climatic conditions. He indicated that in all but arctic and arid conditions, weathering involves both a mechanical break up of the bedrock blocks and chemical changes that act to weaken the blocks. Terzaghi also referenced the Norwegian Geotechnical Institute (Bjerrum and Jorstad, 1957) in their study of long-term rockfall and rockslides related to precipitation. The study reported rockfalls and rockslides between 1650 and 1900. For the period between 1720 and 1760 the frequency of rockfalls and rockslides was 10 times greater than between 1760 and 1810. It was reported that the period from 1720 to 1760 was exceptionally cold and wet and during that period glaciers advanced temporarily. Terzaghi also indicated that during a period of exceptionally wet years the less stable slope areas are "cleaned-off" by rockfalls and rockslides, then many decades may pass before the deterioration of the remaining slopes has advanced far enough to cause a slope failure.

Other processes such as chemical weathering and breakdown of mineral constituents contribute to the long term weathering of the bedrock. Exfoliation is another gradual process than can contribute to a rockfall event. The granite outcrops above the highway may undergo long-term stress release or loss of confinement due to long-term erosional downcutting processes that remove the overburden material and subsequently allow the bedrock outcrops to exfoliate over time.

NATURAL BACKSLOPE EVALUATION USING MODIFIED Q-SYSTEM

Evaluation of the rock mass quality of the bedrock located on the natural undisturbed slopes above the present cutslopes adjacent to I-70 was done using a modified Q-rating system based on evaluation of seismic rockfall susceptibility by Harp and Noble (1993). They recognized that evaluating slope stability for an entire mountainous slope or characterizing the stability of rock slopes on a regional basis is generally beyond the capabilities of standard rock slope stability analyses. Therefore the Q-Rating System, which was previously developed by Barton (1974) for tunneling support design and cost estimation in mining, was modified to provide a relative rock mass quality rating. This system was chosen for the Georgetown Incline since it was not feasible to access the natural steep slopes above I-70 since the area is prohibitively large and walking on the slopes potentially could trigger a rockfall. The cutslopes adjacent to I-70 were rated using the Colorado Rockfall Hazard Rating System (CRHRS) (Andrew, 1994) and the results are not included in this paper. It is important to distinguish between the two areas and rating systems. In general, the CRHRS system was only applied to the highway cutslopes and the Q-rating system was only applied to the natural slopes above the cutslopes.

The following equation illustrates the modified Q-Rating system methodology:

$$Q = \left[\frac{RQD}{J_n}\right] * \left[\frac{J_r}{J_a}\right] * \left[\frac{Jw}{AF}\right]$$

The six factors used to calculate Q are rock quality designation (RQD), joint set number (Jn), joint roughness number (Jr), joint alteration number (Ja), joint water reduction (Jw), and aperture factor (AF). Aperture factor replaces the Stress Reduction Factor (SRF) in the original Q rating system by Barton. Each factor has an associated rating for varying conditions. A description of each rating for the parameters of the modified Q system is included in Figure 5. The RQD is usually measured from core obtained from diamond drilling, however since drilling was not conducted or feasible at the Georgetown Incline, RQD was estimated by observation of the joints for each outcrop and rating them in a relative manner.

Most of the Q ratings were done from across the valley using binoculars and telescopes. For most of the bedrock outcrops on the Silver Plume end of the project site at least two and sometimes three ratings were performed at differing angles on the same bedrock outcrop in an attempt to average the overall rating. Some rock outcrops only have one rating since it was not possible to view alternate angles. Overall, the lower the Q rating the higher the potential of rockfall from a bedrock outcrop. Figure 6 illustrates the approximate location of the larger bedrock outcrops that have been designated with an identification number and rated according to the modified Q system. Figures 7, 8, and 9 illustrate oblique aerial photographs that depict the approximate locations of the mapped bedrock outcrops and chute locations. Table 1 illustrates the corresponding Q rating for each bedrock outcrop.

Rating Category	Rating	Notes
1. Rock Quality Designation	**RQD**	Where RQD is reported or measured as = 10 (including 0) a nominal value of 10 is used to evaluate\ate Q. RQD intervals of 5, i.e. 100,95,90, etc. are sufficient.
Very Poor	0-25	
Poor	25-50	
Fair	50-75	
Good	75-90	
Excellent	90-100	
2. Joint Set Number	**J_n**	For intersections (3 x Jn)
Massive, no or few joints	0.5-1.0	
One joint set	2	
One joint set plus random	3	
Two joint sets	4	
Two joint sets plus random	6	
Three joint sets	9	
Three joint sets plus random	12	
Four or more joint sets, random, heavily jointed, "sugar cube", etc.	15	
Crushed rock, earthlike	20	
3. Joint Roughness Number	**J_r**	
a. Rock wall contact and rock wall contact before 10 cm shear		
Discontinuous joints	4	
Rough or irregular, undulating	3	
Smooth, undulating	2	
Slickensided, undulating	1.5	
Rough or irregular, planar	1.5	
Smooth, planar	1	
Slickensided, planar	0.5	
b. No rock wall contact when sheared		
Zone containing clay minerals thick enough to prevent rock wall contact	1	
sandy, gravelly or crushed zone thick enough to prevent rock wall contact	1	

4. Joint Alteration Number	J_a	ϕ_r (approx.)	Notes
a. Rock wall contact			Values of ϕ_r are intended as an approx. guide to the mineralogical properties of the alteration products, if present.
Tightly healed, hard, non-softening, impermeable filling, i.e.. quartz or epidote	0.75		
Unaltered joint walls, surface only	1	20-35	
Slightly altered joint walls, non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc.	2	25-30	
Silty-or sandy coatings, small clay -fraction (non-softening)	3	20-25	
Softening or low-friction clay mineral coatings (Discontinuities coatings , 1-2 mm or less)	4	8-16	

Rating Category	Rating	Notes
5. Joint Water Reduction Factor	**J_w**	
Dry	1	
Seeps present < or = to 5 gpm	.7	
Well defined seeps> 5 gpm, < or = to 10 gpm	.3	
Well established groundwater flow > 10 gpm	.1	
6. Joint Aperture	**AF**	If perched or loose rocks are common, increase by one. If pervasive joints dip out of slope, increase by one.
All joints tight	1	
Most joints tight, a few open as much as 2 cm (<1 in)	2.5	
Most joints tight, a few loose, open as much as 5 cm (2 in)	5	
Significantly (20 percent) open, as much as 10 cm (4 in)	7.5	
Greatly (60 percent) open, as much as 20 cm (8 in)	10	
Gaping open, many joints open > 20 cm	15	

$Q = (RQD/J_n) \times (J_r/J_a) \times (J_w/AF)$

Figure 5. General Description of Modified Q Rock Rating System.

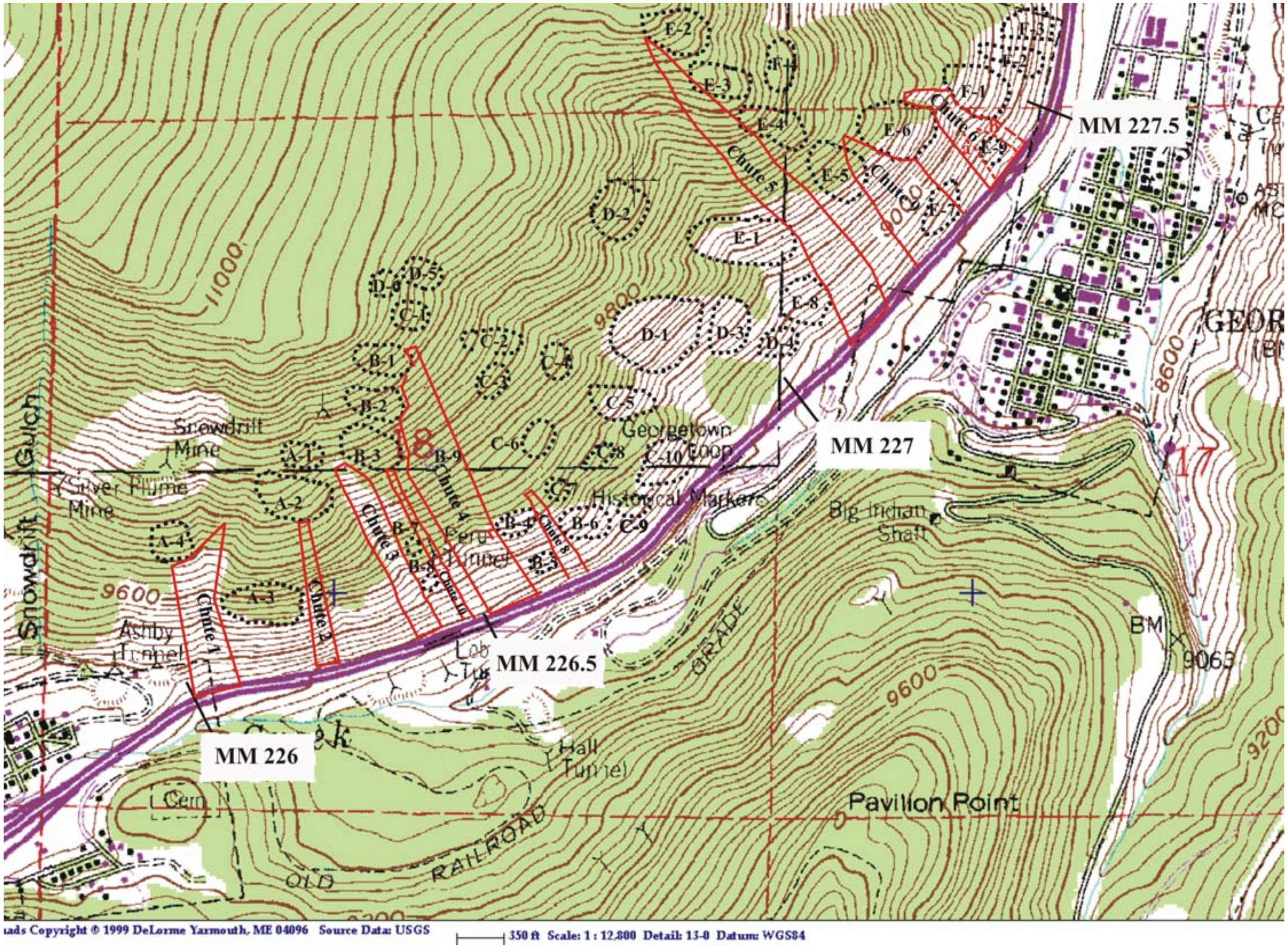
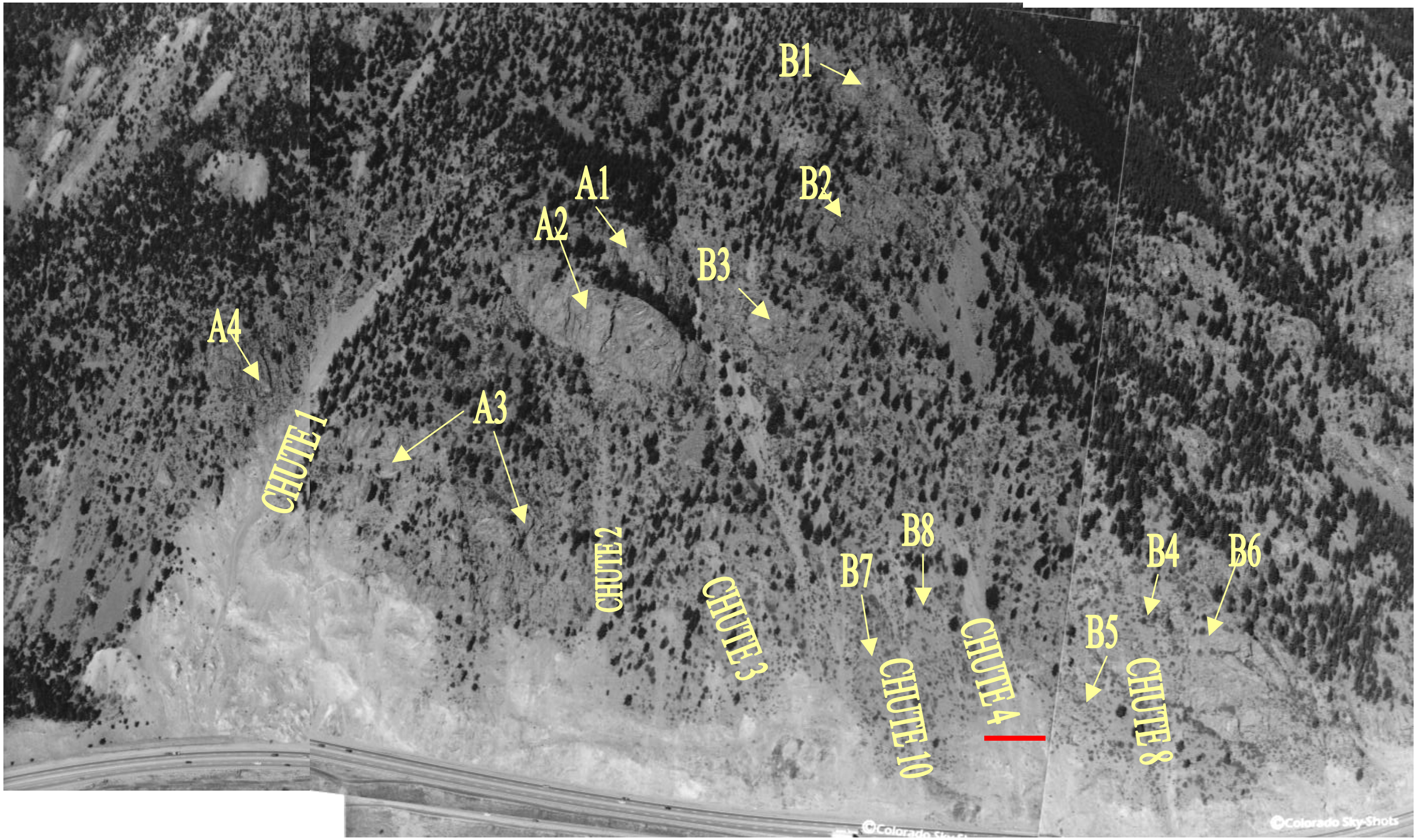


Figure 6. Site map illustrating approximate locations of Q rated outcrops and chutes.



Approximate Location of Existing Rockfall Fence

Figure 7. Oblique photo of Georgetown Incline showing bedrock outcrops and chutes between MM 226 and 226.7.

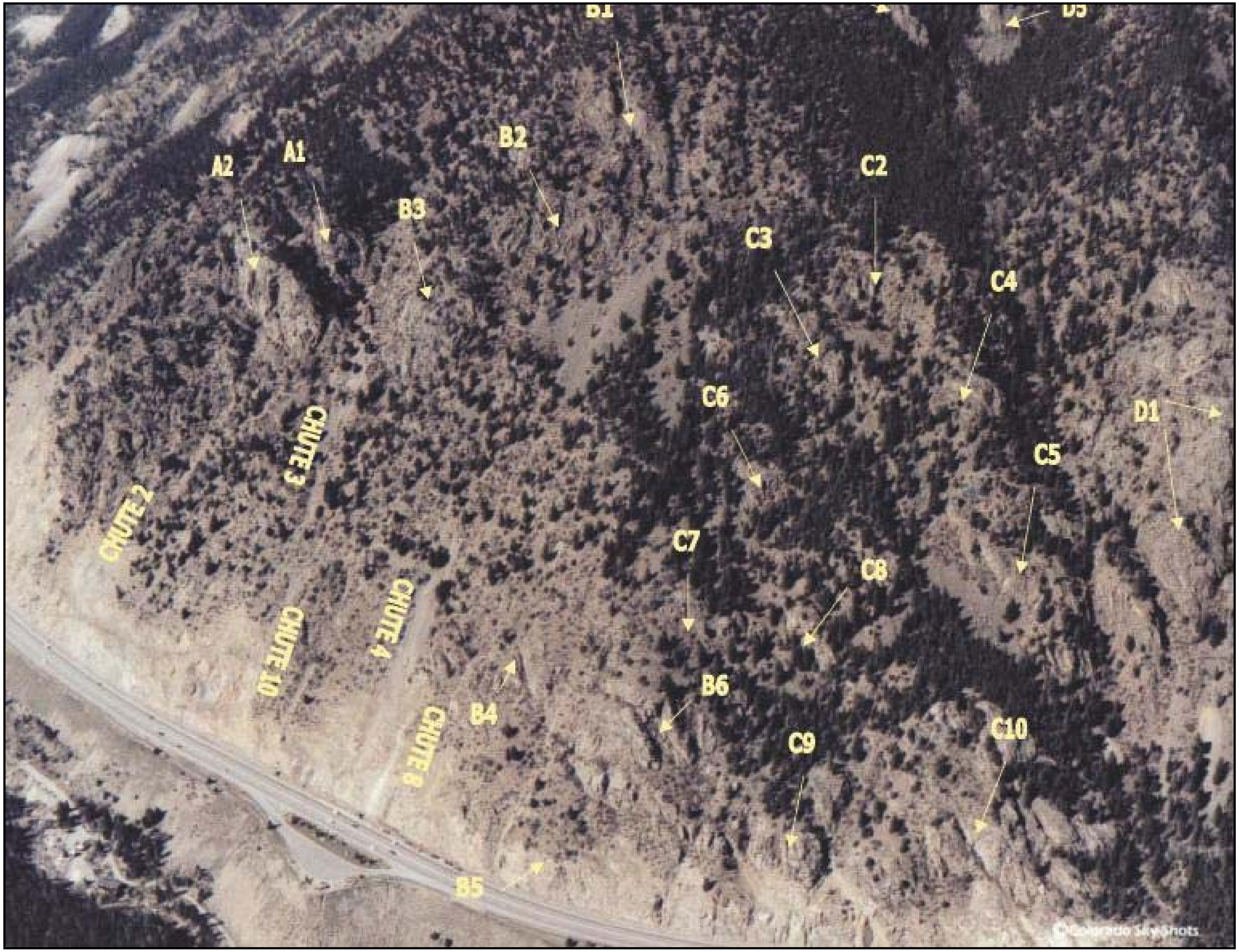
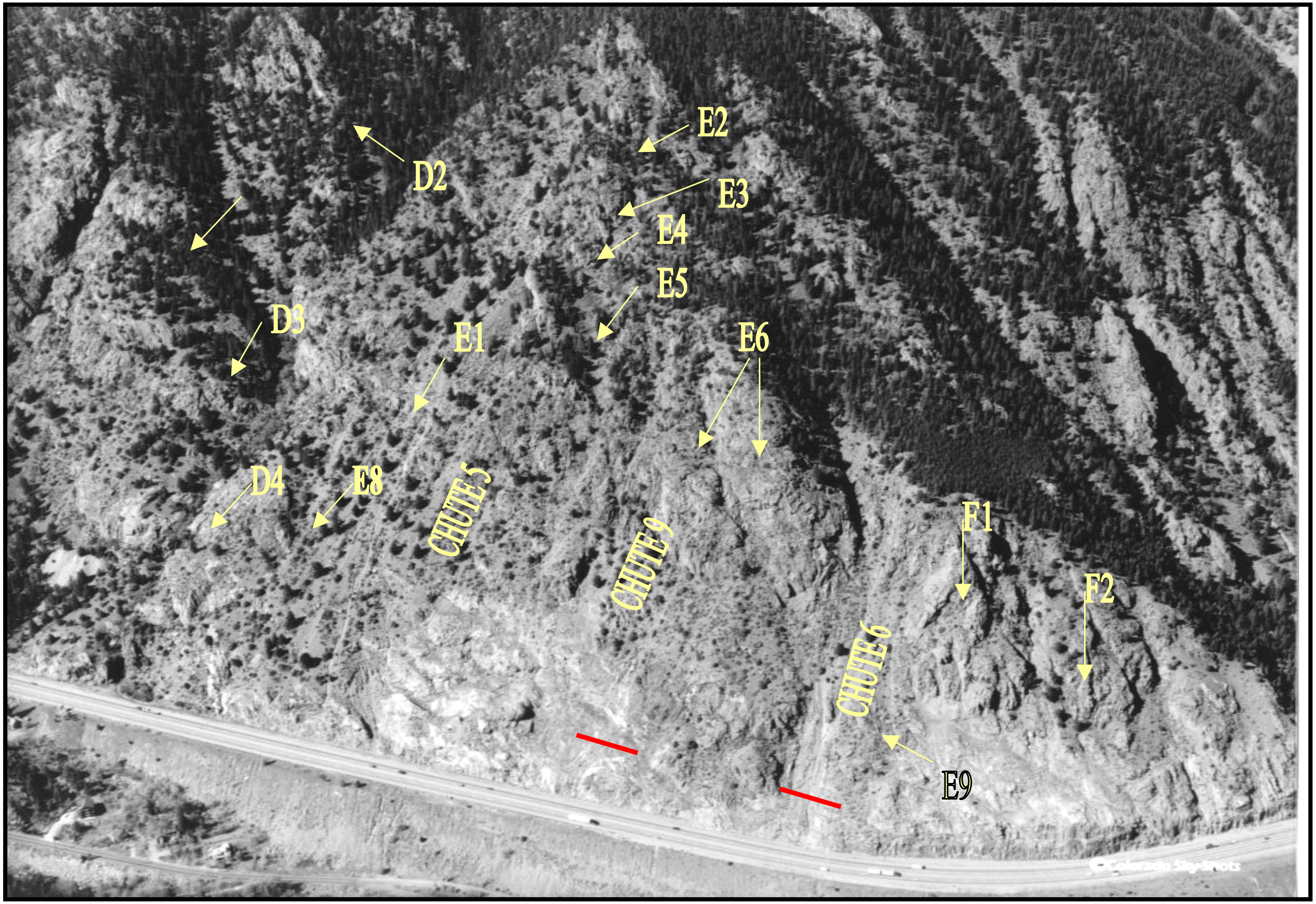


Figure 8. Oblique photo of Georgetown Incline showing bedrock outcrops and chutes between MM 226 and 226.9.



Approximate Location of Existing Rockfall Fence

Figure 9. Oblique photo of Georgetown Incline showing bedrock outcrops and chutes between MM 227 and 227.5.

ROCKFALL ANALYSIS OF SELECTED AREAS

Based on the field review during the modified Q-rating of the bedrock outcrops, several rockfall chutes were identified that typically provided pathways from the top of the natural backslopes to the top of the cutslopes . Selected chutes were modeled for rockfall using the Colorado Rockfall Simulation Program (CRSP) to evaluate the velocities and energy of a potential rockfall initiating from the top of these chutes. Figure 10 illustrates the approximate location of CRSP runs modeled. The profiles for the chutes were approximated by using topographical information from USGS quadrangle maps and other limited sources. This analysis was done to approximate a rockfall for a 2 ft (0.6 m) diameter rock from the top of a designated chute. This provided a preliminary analysis of the energies associated with rockfall, however further study should be performed for pending rockfall mitigative action and determination of more appropriate rockfall parameters. Table 2 illustrates CRSP analysis of rockfall velocities and energies for a 2 ft (0.6 m) diameter rock at approximately the top of the cutslopes above I-70 and at the edge of pavement of westbound I-70.

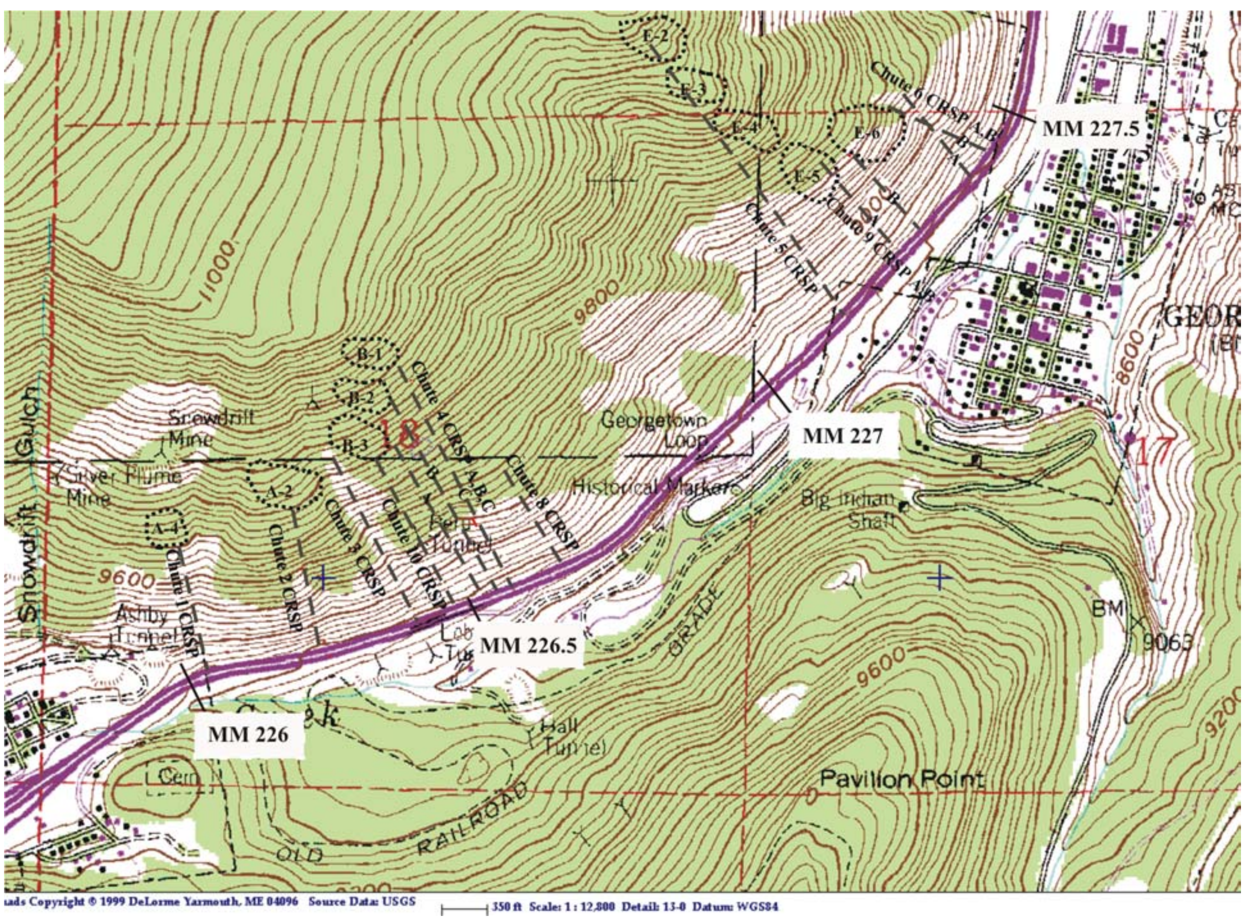


Figure 10. Approximate locations of selected CRSP runs along the Georgetown Inline.

DISCUSSION

Overall, the project reviewed processes that contribute to the potential for rockfall. The most notable physical causes and triggers that initiate rockfall include freeze-thaw activity, which historically appears to be most active during the spring months of March, April, and May. The study also rated the bedrock outcrops on the natural slopes above the current I-70 alignment with the Q-rating system as suggested by Harp and Noble (1993). It appears that many bedrock outcrops with Q ratings of one (1) or less that are associated with chutes have the potential for rockfall reaching the highway. It should be noted that even though the Q rating provides a relative rating scale, rockfall can occur at any time and at any location along the Georgetown Incline and cannot be predicted. Selected chutes were also modeled using CRSP in an attempt to determine preliminary energies, bounce heights, and velocities of a 2 ft (0.6 m) diameter rolling rock (refer to Tables 1 and 2).

The intent of this study was to provide CDOT with a relative rating of the exposed bedrock outcrops on the natural slopes above I-70. CDOT could then evaluate the project site and place rockfall mitigation. The study does not attempt to predict or forecast a rockfall event but rather provides an evaluation tool for rating areas for rockfall potential.

During the duration of this study, rockfall mitigation fences were installed at three sites along the project on chutes with past rockfall activity or overall low Q ratings for the exposed bedrock outcrops. Rockfall mitigation fences typically are designed to catch a range of rockfall impact energies from 25 ft-tons to 250 ft-tons (50,000 ft-lbs to 500,000 ft-lbs) (68 kJ to 678 kJ). Since they are flexible, they absorb and dissipate energy. Vertically placed steel beam posts are placed to support wire rope or ring fence panels. Friction brakes, which are incorporated into the wire rope support system, are also used to dissipate rockfall energy. Draped cable net was used in combination with the fence. The draped cable net is attached to the bottom of the fence panel. Typically an approximate 3 ft (0.9 m) gap between the fence panel and draped net is designed to allow the rock boulders to migrate underneath the draped cable net/mesh to the catchment ditch below that can be cleaned out periodically. Figure 11 illustrates the profile for a typical rockfall fence system with the draped cable / mesh. Rockfall fences with draped cable / mesh installed on the Georgetown Incline ranged from 80 ft-ton (216 kJ) cable-net fence systems to 180 ft-ton (488 kJ) ring-net fence systems. The approximate locations of the rockfall mitigation fences are provided on Figures 7 and 9.

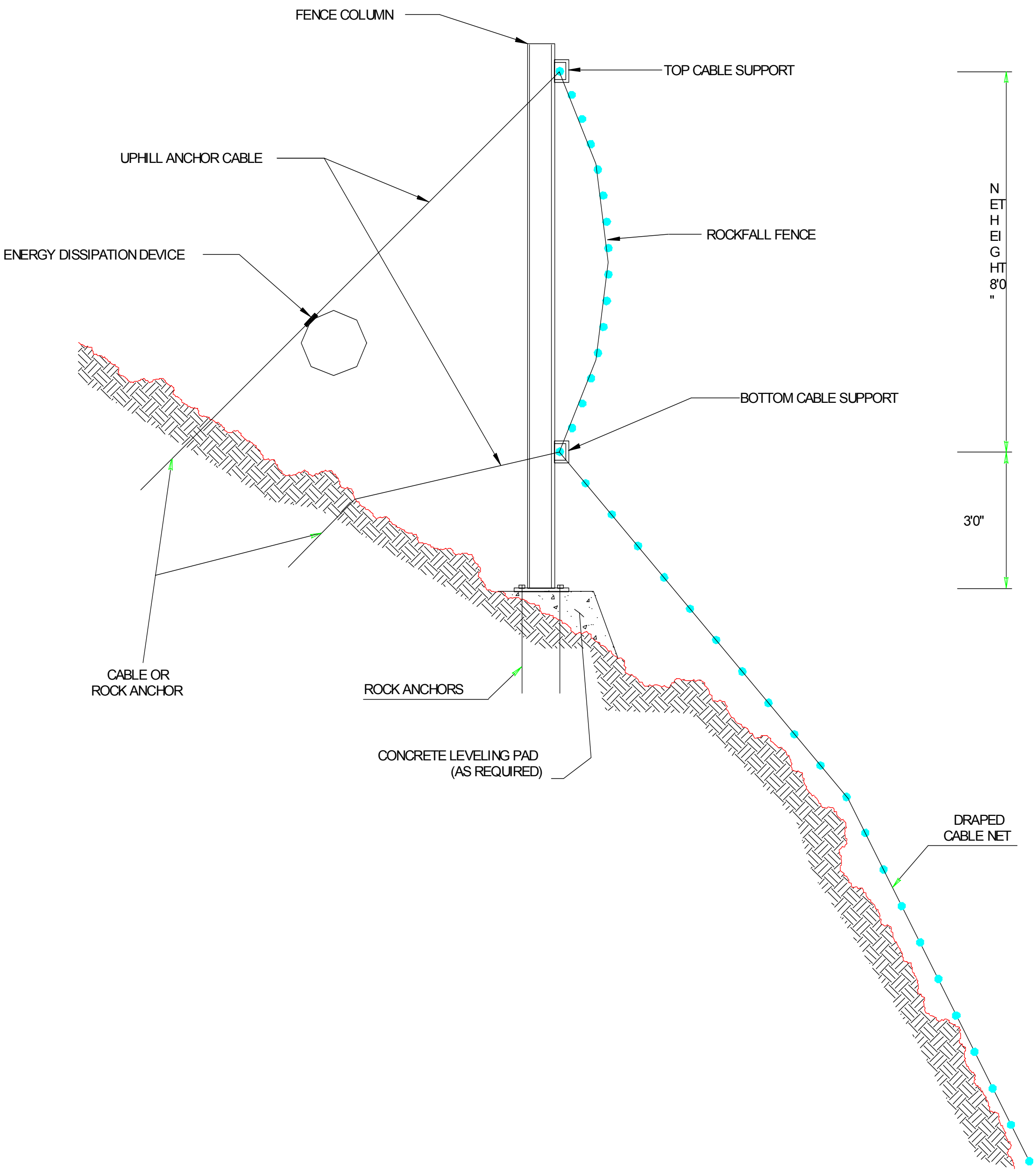


Figure 11. Profile of typical rockfall mitigation fence with draped cable net.

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Table 1. Q-ratings with selected chutes.

Chute	Mile Range	Outcrops	Average Respective Ratings					
1	226.0-226.06	A3,A4	A3=0.50	A4=1.78				
2	226.17-226.29	A1,A2	A1=2.86	A2=1.66				
3	226.36-226.39	A1,A2,B3,B7,B8	A1=2.86	A2=1.66	B3=1.05	B7=3.50	B8=1.73	
4	226.46-226.57	B1,B2,B3,B4	B1=0.52	B2=0.67	B3=1.05	B4=1.71		
5	227.16-227.22	E1,E2,E3,E4,E5,E8	E1=2.22	E2=1.11	E3=0.56	E4=0.56	E5=1.67	E8=5.00
6	227.37-227.44	E6,E7,E9,F1,F4	E6=4.00	E7=1.35	E9=2.50	F1=0.50	F4=0.59	
8	226.6-226.64	B4,B6,C7	B4=0.92	B6=2.15	C7=0.94			
9	227.25-227.32	E1,E2,E3,E5,E6	E1=2.22	E2=1.11	E3=0.56	E5=1.67	E6=4.00	
10	226.42-226.44	B3,B7,B8	B3=1.05	B7=3.50	B8=1.73			

Outcrops	Mile Marker Range	Average Respective Ratings					
A1,A2	226.12-226.36	A1=2.86	A2=1.66				
A3	226.06-226.19	A3=0.50					
A4	226.0-226.05	A4=1.78					
B1,B2,B3,B7,B8	226.36-226.57	B1=0.52	B2=0.67	B3=1.05	B7=3.50	B8=1.73	
B4,B5	226.5-226.6	B4=1.71	B5=2.73				
B6	226.64-226.67	B6=2.15					
C	226.58-226.83	C=1.09					
D's	226.83-226.92	D=0.78					
E1,E8	226.92-227.16	E1=2.22	E8=5.00				
E2,E3,E4,E5	227.22-227.25	E2=1.11	E3=0.56	E4=0.56	E5=1.67		
E6,E7,F4	227.32-227.37	E6=4.00	E7=1.35	F4=0.59			
E9	227.37-227.44	E9=2.50					
F1,F2	227.44-227.63	F1=0.42	F2=0.92				

Note: outcrops that do not have a path to highway were not included

Table 2. Summary of CRSP analysis.

	Approximate Top of Cutslope						Approximate Edge of Roadway					
Selected Chute	Max Velocity (ft/sec)	Ave Velocity (ft/sec)	Max Bounce Height (ft)	Ave Bounce Height ft	Max Kinetic Energy (ft-lbs)	Ave Kinetic Energy (ft-lbs)	Max Velocity (ft/sec)	Ave Velocity (ft/sec)	Max Bounce Height (ft)	Ave Bounce Height ft	Max Kinetic Energy (ft-lbs)	Ave Kinetic Energy (ft-lbs)
1	91	58	22	7	105491	47964	104	39	27	3	137559	32350
2	101	55	15	5	124221	42920	116	71	46	25	160857	65450
3	98	73	35	12	119585	71167	108	52	26	6	140556	49730
4a	74	53	17	5	70189	39198	63	23	6	1	51032	9877
4b	73	56	17	6	63300	41909	107	78	57	25	129227	80328
4c	94	76	124	95	106265	70097	115	42	42	4	153144	34702
5	73	61	11	4	66847	51467	101	61	32	8	123520	61993
6a	103	76	37	12	127692	75123	110	40	21	2	143919	36247
6b	115	77	34	13	155158	79616	120	80	47	20	172041	89687
8	61	49	11	3	49081	34268	74	58	25	12	68212	45704
9a	119	71	32	11	169672	68414	123	64	39	13	178347	64561
9b	100	64	22	8	126556	55924	101	50	44	9	127209	47584
10	100	72	27	11	121043	68269	98	40	13	3	116798	29647

THE EVALUATION OF GEOLOGIC HAZARDS FOR THE I – 70 CORRIDOR PROGRAMMATIC ENVIRONMENTAL IMPACT STUDY

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Key Terms: geologic hazard, landslide, rockfall, debris flow, avalanche, highway, fixed guideway transit

ABSTRACT

The continued increase in Colorado's population and the popularity of recreational activities west of Denver has led to detailed studies to improve the transportation capacity and safety along the I-70 corridor. This corridor is the primary access route through the mountains of Colorado for trucking industries as well as recreation and tourist activities.

The role of the engineering geologist for these studies was to perform the geotechnical, geological, and hazardous materials evaluation and determine their potential impact on the proposed improvements. Due to the effects of the improvements on the existing environment, a programmatic environmental impact study of the corridor was conducted from Denver to Glenwood Springs. The study involved identifying and describing the geologic and soil conditions along the highway, reviewing the potential hazards and engineering constraints and evaluating their impacts to the present highway and future improvements.

The geologic hazards identified include rockfall, debris flows, landslides, avalanches, collapsible and swelling soils, and the hazards related to past mining activity such as heavy metals in mill tailings and mine subsidence. The study area varied but generally encompassed the I-70 corridor and the ground to the ridgelines on either side. Through research of existing maps, aerial photos, and site-specific studies, a geologic hazards inventory map was produced.

A second phase of research involved observation of each geologic condition that affects the existing I-70 corridor. Preliminary field verification was conducted to verify the hazards and their impacts on current and proposed alignments. Evaluation included the history (cause, origin) and the current condition (size, materials, stability) of the hazard as well as any mitigation procedures previously performed. In addition, alternative routes for tunnels were investigated for future expansion of the highway and/or railway.

INTRODUCTION

The primary role of the engineering geologist on the I-70 Corridor study was to provide information on the geology, geologic hazards and mining related hazards and their potential effect on future transportation improvements. The engineering geologist fills the central role between the geological sciences and engineering disciplines, ensuring that the correct geological data is collected and appropriately interpreted in relation to the particular engineering question (Edwards, 1972). This relationship is illustrated by Hoek (1970) who stated that " As a result of the traditional lack of communication between engineers and geologists, the engineer frequently approaches a rock slope with an inadequate understanding of the dominant role of structural geology... the geologist... is unable to recognize those structural features which have engineering significance". Previous projects constructed along the corridor failed to recognize the importance of this input resulting in cost overruns, construction delays and increased maintenance cost. This study differs in substance and the information gathered was used to evaluate potential geologic related impacts on alignments, areas of disturbance and various modes of transportation.

Several transportation improvements have been considered to ease current congestion and provide long-term solutions to mobility and safety. These systems range from minimal interchange improvements to widening the existing interstate to six lanes. Also included were several mass transit systems in the fixed guideway family and buses operating in dedicated lanes. All of these various transportation modes were reviewed and their potential effect on the existing geologic environment was evaluated.

MITIGATION ON PREVIOUS PROJECTS

The transportation facility along the I-70 Mountain Corridor has undergone numerous modifications through the years. Many of the early projects did little or nothing to mitigate existing geologic hazards and soil erosion. In fact, through the design of many of these early projects, some naturally occurring hazards were exposed. More recent projects have incorporated design features that have mitigated the geologic hazards and soil loss. Projects such as the I-70 Glenwood Canyon Project, the Vail Pass projects and the Berthoud Pass projects have utilized excavation and landscaping techniques to minimize soil loss and reclaim existing erosion problems. In addition, the roadway geometry on these projects was designed to minimize slope excavation and follow much of the natural topography.

During construction of the Glenwood Canyon Project excavations utilized rock-sculpting techniques. Rock Sculpting is the blasting of rock by using the existing rock structure to control over-break and blast damage, while creating a more natural appearing rock cut. This technique, originally developed on Vail Pass during construction of I-70, has been used on other projects in Colorado and throughout the western United States.

Other projects have been recently constructed to remediate erosion problems and geologic hazards that were inherent in the original design and construction of I-70. The Straight Creek erosion control projects along the west approach to the Eisenhower-Johnson Memorial Tunnel have been constructed to mitigate the soil loss originating from the over-steepened cut slopes. Rockfall mitigation projects and scaling programs have been implemented at several locations including Dowd Canyon and the Georgetown Incline. These mitigation measures installed along the Georgetown Incline specifically address rockfall originating from the surrounding cut slope and area of disturbance from the Interstate construction.

OVERVIEW OF ISSUES, REGULATIONS, AND METHODS

Issues Identified Through Scoping

Several issues relating to the geologic conditions along the corridor were reviewed, identifying those several key issues that either have a direct effect on the existing transportation system or may impact future improvements. These issues are as follows:

- Potential to exacerbate the existing geologic hazards in the Corridor and negatively impact safety, service, and mobility due to rockfalls, debris/mudflows, avalanches, landslides, and other hazards
- Potential to intersect areas of geologic instability (adverse jointing fracture patterns and/or bedding) and create geologic hazards
- Engineering constraints controlled by limitations on a stable slope angle and revegetation or reclamation potential
- Soil loss due to the increased footprint of the transportation system
- Potential to alter the appearance of the natural setting through the excavation of rock and other subsurface material

The issues were the basis for the evaluation of all corridor improvements under consideration.

Regulatory Requirements

There is very little adopted policy that pertains directly to geologic hazards and the effect on transportation systems in the State of Colorado. Most counties and municipalities have adopted geologic hazard ordinances, requiring proposed mitigation alternatives to be developed as part of construction of new facilities. These ordinances more specifically pertain to residential and commercial construction as required by HB 1041 and SB 35.

Erosion can lead to sedimentation in surface drainages, streams, and rivers if proper erosion controls are not implemented. Polluting waters of the United States is regulated under the Clean Water Act (1972). “Nonpoint sources” are more stringently regulated in the 1987 amendments in section 319. A permit is required for the discharge of a pollutant (sand and gravel) and mitigation and erosion control measures are required. Issues related to soils and soil loss are regulated locally by the Colorado Department of Public Health and the Environment.

The Farmland Protection Policy Act (7 USC 4201 et seq.) directs federal agencies to identify and quantify adverse impacts of the proposed action on farmland. The Act’s purpose is to minimize conversion of agricultural land to non-agricultural uses.

Literature Review

Boundaries for the study area were drawn to include the encompassing valleys as well as all the contributing drainages on either side of the valley. In order to keep the extent of the study area reasonable, only a short distance into each tributary was included. Data on geologic impacts was collected along the extent of the study area and recorded on topographic base maps. Sources of information included geologic and land use maps, research papers and publications, site-specific studies, construction and mitigation reports, field trip guides, aerial photographs, and interviews with local engineers and geologists. Figure 1 is from the Colorado Landslide Hazard Mitigation Plan (Jochim et al., 1988), one of many references used in the compilation of previous studies.

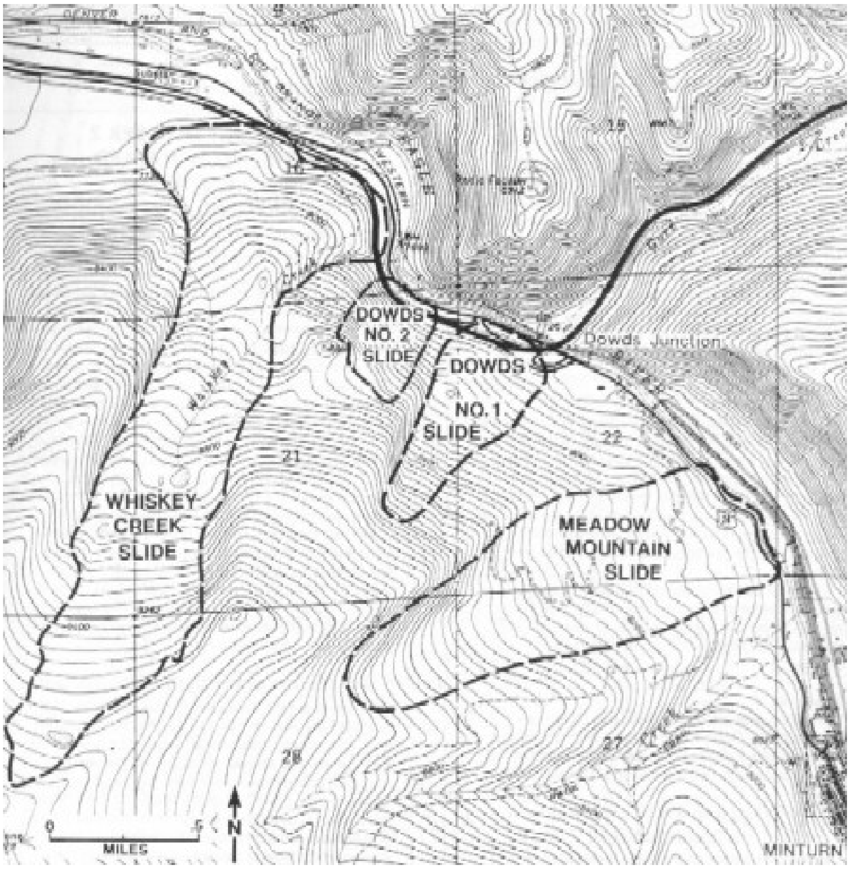

Figure 1. Whiskey Creek Landslide Complex (Jochim et al., 1988.)

For information that seemed to have a high level of dependability, the geologic hazards were outlined and colored. For information that was questionable, the map was appropriately noted for future field review. Features that were very uncertain were labeled, but left uncolored. The basis for discriminating between sets of data was based on the author's knowledge of the study area.

Near the completion of the first phase, aerial photographs were consulted to verify the information gathered, as well as identify new features. Ratings were developed for the existing geologic hazards to evaluate the severity of the disturbed area as described in detail below. Criteria included the influence of climate, proximity to I-70, history of occurrence, and impact of occurrence on transportation and mobility.

The Natural Resources Conservation Service (NRCS) and the United States Forest Service (USFS) have mapped and characterized the soils through the corridor from Denver to Glenwood

Springs for erosion susceptibility. The soils are detailed in the existing maps, database, and reports. Some soil reports are still in draft form, however. The NRCS provided K and T factors, erodibility groups as well as verbal descriptions in the soil description. The USFS provided erodibility descriptions in the soil use and management considerations. Using the NRCS and USFS information and known conditions in the corridor the generalized descriptions are rated to slight, moderate or severe susceptibility to erosion.

Field Checking

The second phase of the study involved field checking the results of the literature review. A staff geologist visually inspected each geologic hazard on the maps, verifying the type of hazard and adjusting the boundaries where necessary. More attention was given to accurately assessing the nature of the hazard than to defining the boundaries in detail. Features that did not concur with their original designation were changed. In some locations, features were eliminated and reclassified as one of the depositional features defined in this study (i.e. alluvial fan, colluvium/ slope wash, soil creep).

Areas marked as "Undifferentiated Hazards" from previous mapping programs conducted by the United States Geological Survey were also field checked to more appropriately define the hazard. During this process the geologic hazard was properly classified and identified on the corresponding map. However, if a recognizable geologic hazard or related features could be discerned, then the area was not mapped or designated a geologic hazard.

GEOLOGIC SETTING

A wide range of geologic conditions are represented and exposed along the corridor due to the vast amount of time represented in the multiple rock formations. The geologic time reflected along the Corridor ranges from recent river, debris and mudflow deposits to Precambrian rocks between 1 and 2 billion years old. Most of the mountainous and rugged terrain in the Rocky Mountains formed during a mountain building orogeny ~72 million years ago that lasted ~7 million years. Numerous faults and folds are present along the Corridor depicting the extensive tectonic episodes.

The multiple sedimentary units represented along the corridor were derived from erosion of a mountain range that pre-dates the present Rocky Mountains, as well as from numerous inland sea advances and retreats. The formations that remain from the inland sea sequences include shale deposits representing shallow sea environments, sandstone and quartzite deposits representing beach environments, and limestone deposits representing offshore coral reefs.

The present topography represents 20,000 years of erosion during and following the last glacial period. With the notable exception of widespread glaciers, the processes that impose hazards to our activities today have been active during this 20,000-year span.

Most of the present configuration of valleys, mountains and canyons seen along the Corridor were shaped either directly or indirectly through alpine glaciation. Periods of glaciation are difficult to identify prior to 100,000 years age (pre-Bull Lake) due to the continual erosion of the loose sediments. Glacial deposits ranging from Bull Lake to more recent Pinedale (10,000 years) have been mapped and dated. Cirques and the U-shaped valleys are some of the remaining features associated with the episodes of glaciation. Following the periods of glaciation, most the valleys visible today have been further shaped by stream and river cutting that formed the classic V-shape at the valley's bottom. Further influences from rain, snowmelt and wind have created deposits of talus and alluvial fans.

GEOLOGIC HAZARDS

The geologic structure, slope configuration, precipitation, wind, and extreme temperature fluctuations all contribute to geologic hazards along the Corridor. The climate includes wetting and drying, precipitation, freeze-thaw, and snowmelt. The slopes can be highly susceptible to erosion with little vegetation cover on most of the slopes.

A geologic hazard is defined as a geologic process that is a risk or potential danger to human life or property (Rahn, 1996). The varied and complex geology and geomorphic processes along the project corridor have led to the development of the several zones of instability and marginal subsurface material. Although a natural process, these features can pose risks to humans either directly by an encounter with the hazard or indirectly through effect of the hazard on roadways, railways or buildings. Conditions that may adversely affect the humans and/or the proposed improvements in the corridor include: faults, adverse rock structure, poor rock quality, and existing geologic hazards (debris/mudflows, rockfall, landslides, avalanche, and collapsible soils).

Landslides include the movement of both competent material (rock slides) and incompetent material (debris slides). Debris flows and rockfall are subcategories of landslides, but were distinguished because of differing modes of failure. Landslides are differentiated from debris flows by the method and means of transport of the material. Unlike flows, slides occur along a defined failure surface, and typically have a defined scarp at the head forming a step like appearance.

South facing slopes along the Corridor are typically characterized by a lack of vegetation, and thin, dry soil. These conditions are more susceptible to relatively frequent yet smaller scale debris flows and rockfall. Landslides and large-scale debris flows are more common on north facing slopes that are characterized by dense vegetation cover, a thick soil mantle and higher soil moisture. The exception is The Watrous Gulch debris flow that occurred on the south-facing slope of Mount Parnassus (Figure 2). This flow originates from an elevation in excess of 12,000 feet, where exposure contributes to the extreme climatic conditions that cause all of the flows along the Corridor. The frequency of slope failures on north facing slopes is much less than that of south facing slopes for the majority of the debris flows, but the magnitude of any slide is much greater.

The conditions that cause rockfall generally fit into one of two categories. One is the case where joints, bedding planes, or others discontinuities are the dominant structural feature of a rock slope. The other is the case where differential erosion or oversteepened slopes are the dominant condition that causes rockfall (Andrew, 1994). The most active rockfall area identified in the

Figure 2. Watrous Gulch debris flow.

Corridor occurs along the Georgetown Incline (Figure 3). The frequency of rocks impacting the existing highway can occur daily.

Avalanches in the study area occur where a large mass of snow or ice moves rapidly down a mountain slope. Avalanche chutes appear as elongate, narrow, barren scars on a mountainside normal to the strike of the valley. Often, a fan-shaped deposit is found at the base of an avalanche chute. Other avalanches can form during periods of high wind causing the rapid accumulation of snow on cliff faces. The most active avalanche zones occur near the east and west approaches to Eisenhower Tunnel and the west side of Vail Pass in the narrows area.

The hazards identified in the second phase were mapped, but were not field-checked. These include rapid settlement, potential for highly mineralized water in road cuts, and potential for seismic activity. Several mining-related hazards were researched, but were not mapped. These include Potential for Mine Blowouts, Water Contamination from Mine Tailings & Tunnels, and Airborne Particulates from Tailings.

SOILS

There are a large variety of soil types present along the Corridor. Soils are primarily a product of their parent material, climate, ecological system, slope and time. The varied geologic conditions in the corridor provide source material from gneiss, granite, volcanics, sandstone and shales to form colluvium, alluvium and various glacial deposits. The slope angles vary from near

Figure 3. Rockfall Chutes, Georgetown Incline.

horizontal along the valley bottoms, to steep and vertical valley sides on nearly all aspects. The ecological environments range from plains grasslands below 5600 ft (1708 m), montane shrublands from approximately 5500 ft to 10,000 ft (1678 m to 3050 m), pinon-juniper woodlands, montane forests and woodlands (ponderosa pine, Douglas fir, aspen, lodgepole pine, limber bristlecone pine) from 5500 to 9000 ft (1678 to 2745 m), subalpine forests from about 9000 ft (2745 m) to timberline, and alpine tundra above timberline.   Riparian environments exist at all elevations located along the margins of streams, rivers, ponds and lakes.

The amount of soil and productivity loss is relative to the degree of slope, reclamation effort, footprint of impact, and climate as well as the productivity of the soils. Soil loss by erosion is related to the particle distribution and the texture of the soil. The soils in the corridor are typically granular in nature and subject to varying levels of erosion when disturbed. Impacts to highly productive soils such as hay meadows or rangeland may occur around Floyd Hill and west of Vail in the Eagle Valley.

Slope stability and re-vegetation issues will not only be affected by the slope and engineering of the slope, but the climatic conditions, and the level of reclamation effort. Typical challenges for re-vegetating slopes in this region are low available water capacity, low water retention, low inherent fertility, and steep slopes. In addition, at higher elevations the growing season can be as short as 30 growing days per year. At lower elevations the precipitation is typically below 20 inches (500 mm) per year, requiring irrigation to reestablish vegetation.

RATING SYSTEM

This final phase of the study was done concurrently with the field checking. It involved evaluating and rating each geologic hazard based upon its impact on the existing Highway I-70 alignment, specifically how it affects future improvements to the highway. The Colorado Rockfall Hazard Rating System (Andrew, 1994) was used to identify and evaluate existing rockfall hazards. During the program, numerical ratings were given by maintenance personnel to quantify the frequency and/or size of the observed rockfall events. The scale ranged from 0 to 4, with 4 depicting the most frequent rockfall activity. Other hazards were rated based on field characteristics as outlined below.

There are five degrees of severity that make up the overall geologic hazard rating system. SEVERE (DO NOT DISTURB), HIGH, MEDIUM, LOW and NOT PROBLEMATIC. Each classification is defined below. A hazard should meet a majority, not necessarily all, of the criteria listed, and each criterion is weighted differently.

- SEVERE ("A")
    - Impact of hazard on I-70 is so great that any disturbance of the hazard for highway improvements is not recommended

- HIGH ("B")
    - Impact of hazard on highway is great
    - Visual evidence of very recent activity [*frequent recurrence interval*]
        - Landslides/ Rock Slides (LS): head scarps and slumps, young vegetation (small aspens, grasses), tilted fences or utilities, surface is hummocky
        - Debris Flows and Mud flows (DF): known recent flow activity, young or no vegetation, mud cracks on surface
        - Rockfall/ Talus Slopes (RF): highly fractured bedrock, rock face is at steep angle, talus on slope below, no catchment besides the highway, rockfall rating of "4" based on maintenance records (Andrew, 1994)
        - Avalanches (AV): well defined avalanche chute that is deep and contains no trees, limited or no runout zone besides highway
        - Mine Subsidence (MS): known recent subsidence, mines developed in poor quality rock or unconsolidated material
        - Collapsible Soil (CS): Not applicable. Not a high-risk event in the study area.
        - Rapid Settlement (RS): all identified areas. No variation because only applies to a single location along the corridor.
    - If failure,
        - Long-term loss of service to highway-impedance of full roadway, road not traversable or drivers must stop
        - Immediate mitigation needed
    - No mitigation previously done

- MEDIUM ("C")
    - Impact of hazard on highway is moderate
    - Generally less than 500 ft (152 m) from highway
    - Visual evidence of somewhat recent activity
        - LS: no visible scarps or slumps, vegetation is intermediate stages (tall aspens, some evergreen), utilities and fences stand straight, surface is hummocky
        - DF: Debris flow deposit still visible, young vegetation and thick colluvium on surface
        - RF: highly fractured bedrock, little or no talus on slope below, rock face at moderate to steep angle, limited catchment area, rockfall rating of "3" or "2" based on maintenance records (Andrew, 1994)
        - AV: chute is not active every year, runout zone may or may not reach highway
        - MS: Not applicable. Either HIGH or LOW rating
        - CS: known recent collapse of soil, cracks in pavement
        - RS: Not applicable. Only HIGH rating.
    - If failure,
        - Moderate loss of service to highway, impedance to less than half of the roadway, driver must slow down
        - Long-term mitigation needed
        - Periodic maintenance required
    - Some limited or partially effective mitigation may have been done in the past

- LOW ("D")
    - Impact of hazard on highway is minimal
    - Generally more than 500 ft (152 m) from highway (except rockfall)
    - No visual evidence for recent activity
        - LS: no fresh head scarps, vegetation is mature (evergreens), surface is hummocky
        - DF: most north-facing slopes with long recurrence intervals (50-100 yrs)
        - RF: rock is not highly weathered or there is a good catchment area for debris, includes a rating of "1" based on maintenance records (Andrew, 1994)
        - AV: sufficient runout zone, but suspended debris may reach the highway.
        - MS: all areas other than those listed in HIGH, good quality rock
        - CS: all areas that are not included in the MED category. Assume all has the potential to collapse.
        - RS: Not applicable. Only HIGH.
    - If failure,
        - Little or no loss of service to the highway, impedance of shoulder or less, no noticeable pavement damage
        - No mitigation necessary
        - One-time maintenance needed

    - Includes areas where extensive mitigation has been done which is mostly effective

- NOT PROBLEMATIC ("E")
    - Natural hazard does not cause a problem to current or future highway improvements to the existing alignment
    - If failure,
        - No loss of service to the highway
        - No mitigation necessary
        - No maintenance needed
    - Includes areas where extensive mitigation has been done that is highly effective
    - Rockfall rating of "0" or outcrops that have not been rated by The Colorado Rockfall Hazard Rating System.
    - Includes all unmodified "Undifferentiated Hazards" from USGS 1° x 2° map
    - Includes areas that are blocked from the highway by a physical barrier (ridgeline, valley, negative difference in elevation, stream). In this case, the hazard can be within 500 ft (152 m) of I-70.
    - Debris flows cannot be classified as "NOT PROBLEMATIC" unless there is a physical barrier as described above. Theoretically, any of the debris flow channels could reactivate.

RESULTS

The length of I-70 within the project area was divided into 14 geologic domains. An assessment of the geologic hazards was done for each section, based on the data gathered during the study. In addition, each section was given an overall rating for the cumulative effect of all geologic hazards encountered in that particular domain. Table 1 provides a summary of the geologic hazards present in each of the domains and their respective severity index.

Each mapped hazard was rated according to the rating system described above and entered in to the geographical information system ARCinfo. Information regarding the size, type of hazard, estimated recurrence interval and rating were also recorded for each of the mapped hazards. The data them can be easily accessed for evaluating potential impacts future corridor improvements will have, and determining appropriate mitigation alternatives.

Other information provided included a road log for entire length of the corridor. The log highlights problematic areas that will have a significant impact on the design of future highway projects (SEVERE and HIGH designations). Other sites are discussed to clarify the rating designation given to a particular hazard by listing the criteria that were met. This is only a listing of significant features and is not a complete listing of all the geologic hazards encountered within the field area.

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Table 1. Summary of Geologic Conditions and Severity Index for Current Condition in each Domain

Geologic Condition	White River Plateau (MP 119-133)	Eagle Valley (MP 133-157)	Red Sandstone (MP 157-171)	Gore Range (MP 171-195)	Ten Mile Canyon (MP 195-203)	Continental Divide (MP 203-227)	Glaciated Valley (MP 227-234)	Mineralized Zone (MP 234-241)	Rolling Hills (MP 241-259)	Front Range Hogback (MP 259-262)
Adverse Faulting	Low	Low	Low	Medium	Medium	Severe	Medium	Medium	High	Medium
Adverse Rock Structure	Low	Low	Low	High	Severe	Severe	Medium	High	High	Medium
Poor Rock Quality	Low	High	High	Medium	Low	Severe	Low	Low	Medium	Medium
Debris Flow	Low	High	Medium	High	Medium	High	High	Low	--	--
Rockfall	High	Low	Low	High	Medium	Medium	Severe	High	Low	Low
Landslide	Low	Severe	Severe	High	Low	Severe	Low	Low	Severe	Low
Avalanche	--	--	--	High	Medium	High	--	--	--	--
Collapsible Soil	Medium	High	High	--	--	--	--	--	--	--
Rapid Settlement	High	--	--	--	--	Medium	--	--	--	--
Soil Erosion	Moderate	Slight - Severe	Slight - Moderate	Moderate - Severe	Moderate - Severe	Moderate - Severe	Severe	Severe	Moderate - Severe	Severe

-- indicates "not problematic"

MP = "mile post"

ROCKFALL MITIGATION AND SLOPE RECONSTRUCTION AT HIGH ALTITUDES, THE INDEPENDENCE PASS, HIGHWAY 82 'TOP CUT' PROJECT

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Key terms: glacial morphology, rockfall, rockslide

ABSTRACT

The Independence Pass 'Top Cut' is a mile-long corridor of eroding cut slopes along Highway 82 in Pitkin County, Colorado. Just west of Independence Pass, the corridor ranges in elevation from 11,600 to 12,000 ft (3,536 to 3,658 m). The site lies within the core of the Sawatch Range in Precambrian igneous and metamorphic rock terranes. Glacial morphology, adverse geologic conditions, poor early road construction, climate, and past detrimental maintenance procedures have set the stage for serious erosion problems, degraded environmental conditions, and rockfall and rockslide hazards along the Top Cut Corridor.

The Independence Pass Foundation (IPF), a nonprofit environmental organization based in Aspen, formed the Independence Pass Restoration Team Partnership to address the problems, and develop and fund solutions. This partnership includes private individuals, various state and federal agencies, Pitkin County, and the city of Aspen. This group is working to mitigate these problems and restore the eroding slopes. In 1995 an engineering geological study of the Top Cut corridor was conducted that included digital elevation modeling and GIS mapping. Later a hydrologic study was also commissioned for the Top Cut area. These data have been used to select and develop project sites along the Top Cut corridor to mitigate rockfall hazards and reclaim heavily eroded slopes. The primary factors in the design concepts were cost and natural appearance. In addition to internally-funded projects at the site, IPF and Colorado Department of Transportation (CDOT) shared costs in rockfall mitigation and slope reconstruction, restoration, and revegetation projects in 1996-1997, 1998, and 2001-2002 at the Top Cut using statewide rockfall mitigation funds and federal enhancement grants. The challenges of construction and restoration at high altitudes resulted in innovative rock slope stabilization construction techniques, while reducing the visual obtrusiveness of scarred slopes and constructed mitigation structures. The overall project has featured pioneering use of boulder-faced, geotextile-reinforced, slope reconstructions. This technology allowed slope reconstruction to match the existing cut slope scarp at grades that are amenable to revegetation in the harsh high-altitude climate, offering design flexibility and natural appearances suitable for the area. Slope restoration costs were kept down by the use of donated materials and equipment, and inmate crew labor provided by Colorado Department of Corrections.

INTRODUCTION

State Highway 82 crosses Independence Pass at an elevation of 12,093 ft (3,686 m) at the Continental Divide. Originally an early stagecoach route from Leadville, the Independence Pass route was improved for vehicles in 1927 and further modernized in the 1950's. It was the excavation for the modern two-lane corridor that led to the eroding cut slopes and denuded slopes seen today.

In the early design and construction of Highway 82, as in many highways of Colorado, geotechnical and environmental impacts on the long-term erosion and stability of cut slopes was not a policy priority. Once the fragile alpine topsoil was stripped away and the toe of a natural slope is removed, instability results in the oversteepened cut slope. Further erosion of the weathered granitic rock mass occurs and rock and soil debris continually fall from the cut slopes and collect at the base of the slope, many times on the roadway itself. Continued maintenance is then required to dispose of the material. For many years, this material was simply cast over the side of the road embankment, causing additional damage to the native slope below. Figure 1, a photo mosaic taken from across the valley, shows the level of degradation.

Figure 1. Highway 82 Top Cut area near Independence Pass. A switchback of the historic stagecoach road can be seen on the right, below the existing highway.

Severe winter weather and avalanche hazards force the CDOT to close this highway from October until the Memorial Day Holiday weekend every year. Because of its seasonal status and non-use by commuters, Highway 82 rates a low priority for CDOT in the region. Many locals feel that the roadway corridor has been neglected.

IPF was formed in 1989 by the Environmental Research Group, an Aspen area non-profit citizens group started by Robert Lewis, that was dedicated to mitigating human impacts on high mountain ecosystems. The IPF was created to focus on major restoration work along the Independence Pass corridor from Aspen to the Twin Lakes, and to coordinate restoration activities between the many government entities involved in Independence Pass management. In 1990 the IPF formed the Independence Pass Restoration Team, consisting of the IPF, Pitkin

County, the City of Aspen, CDOT, U.S. Forest Service (USFS), Colorado Department of Natural Resources, Colorado Geological Survey (CGS), and other agencies. Volunteers carried out the majority of the early restoration and revegetation work in lower elevation areas. Significant progress was made in several locations between Aspen and the 11,000-ft (3,353-m) elevation level (White and Fuller, 1999). Map Plate 1 is a location map showing the work to date that has been sponsored by IPF.

In 1994, attention was turned to the Top Cut. Some early "stab-in-the-dark" revegetation attempts were made using fertilizers, seed, and hydromulch. These attempts were generally unsuccessful and wasteful given the limited resources available. The ground complexities and climate required further in-depth alignment studies and mapping to determine the geologic and slope conditions. Photogrammetric maps and digital elevation models (DEM) at two-foot contour accuracy were developed and used for engineering geologic, avalanche susceptibility, and hydrologic studies of the Top Cut area. The precise DEM data also enabled detailed design work during subsequent project planning. Prototype projects sites were developed with IPF and CDOT rockfall mitigation funding that utilized various rockmass reinforcement, rockfall protection, slope reconstruction, and revegetation using turf reinforcement matting (TRM) and erosion control matting (ECM) methods. The prototype projects where constructed in 1996 and 1997. Upon further evaluation and as additional funding became available, larger slope reconstruction, rockfall mitigation, and slope restoration projects were designed and constructed in 1998 through 2002. This construction was co-funded by IPF and by CDOT through the Statewide Rockfall Program.

ENGINEERING GEOLOGIC SETTING OF THE TOP CUT

The Independence Pass Top Cut lies at the core of the rugged Sawatch Mountain Range in Pitkin County, 17 miles (27.4 km) east of Aspen and 0.4 miles (0.6 km) west of the Continental Divide. The Continental Divide marks the border of Pitkin and Lake County and is the southeastern boundary of the Mt. Massive Wilderness Area. The land, except the highway right-of-way, is administered by the USFS through the White River National Forest and its District Office in Aspen. Independence Pass stands on the crest of the massive Sawatch uplift. This mountain building occurred during the Late Cretaceous-Early Tertiary Laramide Orogeny (72-40 million BP) and was completed during periodic block faulting and regional uplift in the Late Miocene and Pliocene Epochs (25-5 million BP). During and since the mountain building, millions of years of erosion have stripped away thousands of feet of sedimentary rocks, leaving a core of Precambrian-aged granites, metamorphic rocks, and small discontinuous Tertiary-aged intrusives within the study area. Periodic glacial activity during the Pleistocene age (2 million to 10,000 BP) shaped the mountains and the valleys to what is seen today (Van Loenen, 1985).

The Independence Pass Top Cut is a mile (1.6 km) of road cuts on Highway 82, from milepost 59.7 to 60.7. It is at the tree line, at 11,600 to 12,000 ft (3,536 to 3,658 m) in elevation. The alignment of the roadway leaves the bottom of the Upper Roaring Fork Valley, curves up onto the eastern flank of the valley, and begins to level off as it approaches the Continental Divide (Figure 2). The valley is glacial in origin and has the classic U-shape cross-sectional form. This morphology has resulted in very steep natural slopes through the mid-elevations of the valley.

In many areas slope steepness approaches or exceeds 1:1 (45°) and, on undisturbed sites, natural vegetation has taken hold despite these extreme grades. The geomorphic system is in a state of equilibrium because the topsoil formation and vegetation could follow the retreating ice as the glacier diminished and emptied from the valley some 12,000 years ago. Due to the steepness of the natural slope, substantial cuts and fills were required to facilitate a modern two-lane highway through the steep valley flank. Dis-equilibrium occurred when the roadway was excavated through the upper valley (White, 1996 and White and Fuller, 1998).

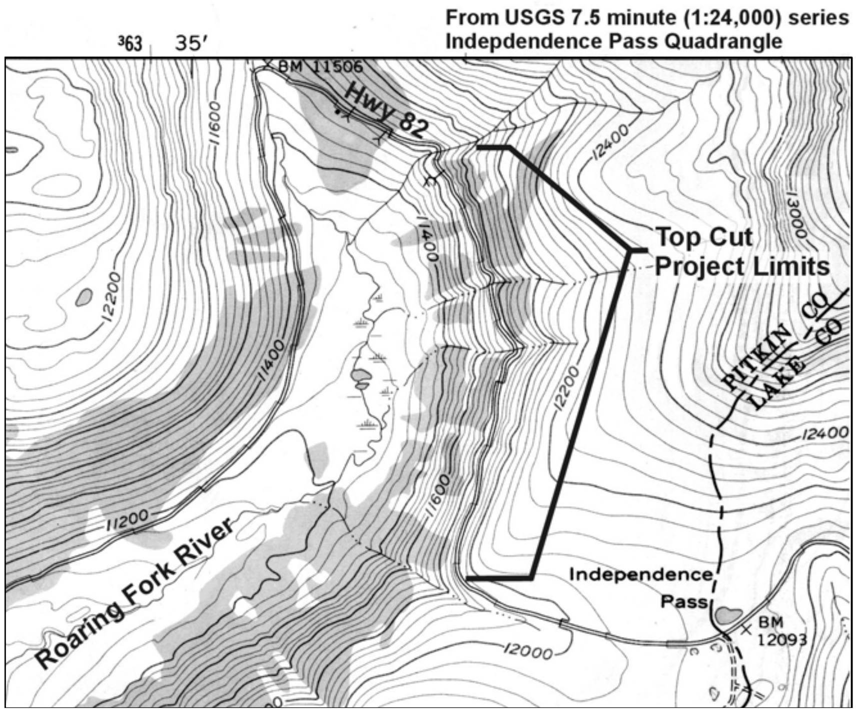

Figure 2. Top Cut Project limits encompass approximately one mile of highway alignment. Contours are in feet.

Variable rock and soil materials were encountered along this alignment that, over the years, have reacted and weathered differentially upon exposure to the alpine climate. In this highway alignment, cuts were excavated in materials that ranged from very hard bedrock, generally resistant to weathering, to highly disintegrated rock and soil that erode easily.

The cut slopes of Highway 82 at the Top Cut expose granitic and metamorphic rocks. These cut slope exposures were mapped by White (1996) using a modified weathered-rock classification. Due to the high elevation, mechanical weathering is the primary erosive agent. Very little actual chemical decomposition of the crystalline rocks was noted, except along isolated, well-delineated shear zones. The extent of weathering is expressed in degrees of granular disintegration. Soil gradation tests of samples collected from the base of the most eroded of the cut slopes, not including the larger rocky material, revealed gravelly sand with less than 10% passing the #200 sieve (Wright Water Engineers, 1998). Listed are some general descriptions of the mapped rock units:

Type 1. **Hard Rock.** Very hard and resistant to erosion. Does not easily backwaste or erode in a rock cut slope. Rock is fresh but certain zones can be highly jointed and fractured. Those discontinuities are clean without in-filling. There can be some grus weathering along prominent joints. Internal rock strength can hold a vertical cut slope except where unfavorably-oriented discontinuities exist. Rock rings when struck with a rock hammer. Modes of failure include planar, wedge, and toppling along pre-existing discontinuities.

Type 2. **Slightly Weathered Rock**. Similar to Type 1 but certain zones are highly fractured and jointed and there are commonly granular disintegration (grus) zones along prominent joints. Weathered zones still retain tight rock fabric and are still resistant to raveling or erosion in cut slopes. Slow granular disintegration of rock creates small grus fans at the base of slopes. Rock thuds when struck with a rock hammer. Modes of failure include planar, wedge, toppling, and raveling (granular disintegration and surface particle detachment).

Type 3. **Weathered Fragmented Rock**. Highly fractured and weathered. Cut slopes are still bedrock but are beginning to crumble and fail, losing rock structure and strength. The more resistant rock zones are heavily fractured. Commonly, zones have disintegrated into grus where "core stones" are prevalent. The rock cannot maintain the original excavated cut slope angle and continually erode back. Debris fans annually occur at base of slopes. Rock is slightly penetrated when struck with a rock hammer. Modes of failure include toppling, raveling, and very shallow and irregular soil-type circular failures.

Type 4. **Disintegrated Rock and Soil**. Highly fractured and very weathered with most rock structure lost. Small areas of relic rock structure are jumbled and indistinct. Cut slopes continuously erode back to an angle of repose that approximates rocky colluvial soil (34°-40°). Slopes steepen where protected by vegetation cover at brow of cut slope. Debris fans at base of slopes require periodic removal to retain road ditch width. Rock is easily penetrated when struck with rock hammer. Modes of failures include raveling and shallow soil-type circular failures near scarp of cutslope.

The mapping of the cut exposures along the Top Cut highway alignment has revealed a close correlation of weak rock zones to advancing erosion into the natural tundra slope, which was predictable. Once the top 6 to 12 in (15 to 30 cm) of topsoil and binding vegetation has been stripped away, erosion quickly removes the remaining thin veneer of rocky colluvial soil. All of the slopes that exhibit the most pronounced erosion and upward advancement of the cutslope brow have had all the topsoil removed and are now eroding into the broken, heavily weathered, bedrock beneath. Unfortunately, this condition results in a rocky soil mantle that is barren of organic material and constitutes a poor growth medium for revegetation efforts (White and Fuller, 1999).

Generally, the original highway cut slopes exceeded the angle where vegetation could recur naturally and no attempt was made to revegetate the slope after excavation. At the design cut angle, and without any vegetation cover or slope stabilization techniques, the surface of very weathered disintegrated rock and residual soil could only support the original cut slope temporarily. Through the years the cuts in the weaker material eroded back as the slope attempted to achieve stability or equilibrium at a reduced grade, near the angle of repose. The original slope grade was stable where vegetation bound and protected the rock and soil. However, backwasting of the denuded cut slope creates an oversteepening situation below the vegetated brow. In some circumstances the oversteepening just below the brow is vertical to even overhanging. This condition is shown in Figure 3. The eventual undermining of the brow of the cut slope will result in shallow slump failures of clumps of top soil and vegetation, and further headward erosion into the above natural vegetation. Figure 4 illustrates the advancing erosion, common of cut slopes at the Top Cut.

Figure 3. Typical raveling brow at highway cut slope at Independence Pass Top Cut.

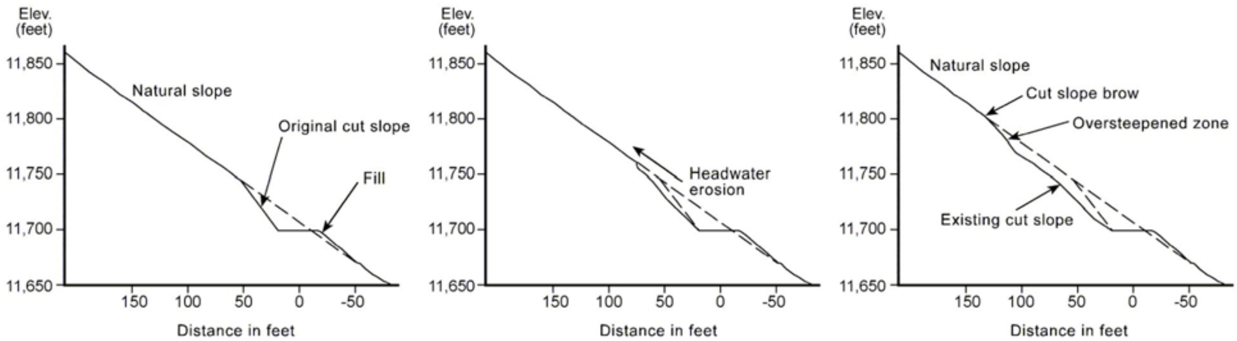

Figure 4. Schematic of raveling failure at cut slope in Type 3 and 4 rock.

Fortunately, the rock along the Top Cut is granitic or metamorphic. Even those slopes that are in Type 4 rock and soil are still in granular material with high frictional strength. Overall, the global stability of these slopes is good and the possibility of traditional deeper-seated rotational failure occurring is highly improbable. Certain areas of the Top Cut where more competent rock is prevalent are prone to planar failures where jointing dips toward the roadway and are day-lighted by the cut slope. The more weathered slope materials are granular and angular and so, while having high frictional strength, have little cohesion. The mode of failure of these slopes is by the process of *raveling*. Raveling of cut slopes in granular soils is the separation, or particle by particle removal, of soil constituents (ranging from sand to boulder-sized rock fragments) from the surface soil or weathered rock mass of the slope. An oversteepened exposure is usually subjected to raveling from normal weathering processes (i.e., rain, snow, wind, freeze-thaw cycles, etc.).

The cut slopes mapped as Type 3 and Type 4 slopes, Weathered Fragmented Rock and Decomposed Rock and Soil, exhibited the worst raveling and shallow failures. The cut slopes mapped as Hard Rock and Slightly Weathered Rock hold the original cut slope more successfully and retain, for the most part, a stable brow of rock where the overlying tundra is not receding. All the slopes erode, albeit at different rates, and fill the ditch below with raveling and rockfall debris, which sometimes enters the roadway. Until recently, the CDOT maintenance method of disposal was to simply cast this material over the side onto the fill slope below the road. This activity, with snow removal also disposed over the side, has created accelerated erosion below the roadway in what is called the Stripped Vegetation Zones. Rills and gullies in the zone feed into small debris fans at the valley bottom near, or into, the Roaring Fork River. As can be seen in figure 1, these accelerated erosion or stripped vegetation zones below the roadway are the most visually obtrusive aspect of the Top Cut as seen from across the valley and create the worst environmental degradation hazard to the Roaring Fork River.

The Independence Pass Top Cut has been recognized for some years as an active rockfall hazard zone. Five slopes have been delineated that have been rated in CDOT's Colorado statewide rockfall project. Two slopes rated sufficiently severe to be included within the top 10 rockfall hazard slopes in CDOT Engineering Region 3/Maintenance Section 2, one of which rated No. 11 overall, statewide (Andrew, 1994). This hazard will worsen every year as weathering continues, traffic volumes continually increase, and vehicle impact probabilities rise.

Avalanche hazards also exist at the Top Cut. While not usually a hazard to the traveling public since the road is closed during the winter, they do occur during late snow storms after the road officially opens for Memorial Day weekend. Tree trim-lines, above and below the roadway, show that large avalanches have occurred at the site. Mears (1979) has mapped several portions of Highway 82 at the Top Cut as high avalanche-hazard zones.

THE INDEPENDENCE PASS TOP CUT PROJECT

As previously stated, to understand the complexity of the geology, soils, hydrology, and climate of the Top Cut area, engineering geology, avalanche, and later hydrologic studies of the area were conducted. In addition, research was conducted on available anchored revegetation systems and on existing CDOT research and construction projects at Interstate 70, west of Eisenhower Tunnel, and Highway 40 on the west side of Berthoud Pass (Goff and Rosener, pers. comm.).

The biggest problems perceived at Independence Pass Top Cut were the continual erosion of (1) the existing cut slopes and (2) the damaged slopes below the road that have been stripped of vegetation, either through erosion or burial. Rates of erosion varied, dependent on the quality of the rock mass. Table 1 shows the mapped rock units from the engineering geology study, their areal extent (calculated from the GIS map), and sediment yields from the hydrologic study (Wright Water Engineers, Inc., 1998).

Table 1. Sediment yields based on mapped zone areas calculated from GIS geologic map, from Wright Water Engineers (1998). Note high yields for Type 4 rock and Stripped Vegetation Zone.

Locations	**Acres**	**Sediment Yield**	**Estimated Sediment Load**
		(tons/acre/year)	(tons/acre/year)
Cut Slope Rock units			
Type 1 (hard rock)	0.89	0	0
Type 2 (slightly weathered rock	1.03	2.84	2.9
Type 3 (weathered fragmented rock	1.93	5.68	11.0
Type 4 (decomposed rock and soils	2.26	32.65	73.8
Natural vegetated slope	0.18	1.66	0.20
		Cut slope total	**88.0**
Basins above cutslopes	574	2.76 (avg.)	1483.0
Slopes below highway			
Stripped Vegetation Zone	20.05	71.4	1432
Exposed bedrock	5.78	0	0
Vegetated zones	145.17	3.3	479
Note: Sediment yields and load figures do not include the larger rocks or boulders eroding from the slopes. CDOT estimates they remove 750 tons of rock annually during ditchline maintenance.			**TOTAL – 3482.0**

Prototype project planning for slope reconstruction, stabilization, and revegetation began in the winter of 1995/1996. The priorities outlined by the IPF were (1) slopes dangerous to the public (rockfall), (2) Slopes with the highest sediment yields and erosion potential, and (3) Slopes that could be treated with the least cost. Out of fiscal necessity, planning centered on cost. The Foundation relies totally on donations, volunteerism, and assistance by local, state, and federal government agencies. During the planning period, a new funding source was found in CDOT's statewide rockfall program. Limited funding was being set aside for Highway Safety Projects for rockfall mitigation at each of the CDOT Region's top 10 rated slopes. Two slopes at Top Cut fell into that category, both of which were being considered as prototype project sites for IPF. The CGS assisted CDOT in defining a scope of work for rockfall mitigation at the Top Cut-rated slopes that included scaling, anchored wire mesh, rockbolts, and anchored revegetation matting.

Twin Gullies Site

CDOT rockfall mitigation funds were devoted to rockfall mitigation such as scaling, rock bolting, and wire mesh at the highest rated slope near the top of the Top Cut. It enabled IPF to devote the bulk of their resources to slope restoration and revegetation. The IPF prototype site selected is called the Twin Gullies. At this site, two small gullies mark the weaker formational contact of granite, gneiss, and pegmatite. Gullying at these near-vertical rock contacts had advanced up the steep hillside more than 130 ft (39.6 m). Fans of debris accumulated annually in the ditch below these gullies and included detached slabs of tundra topsoil.

Several options for mitigation and stabilization of the Twin Gullies were examined including tiered retaining walls, oversteepened slope designs, draped wire mesh, and pneumatically sprayed concrete (shotcrete). The differing techniques were weighed regarding costs, natural appearance, and ability to revegetate. The IPF Board Members felt that reconstruction and revegetation of the eroded slopes to the greatest possible extent would be most in keeping with their mission and purpose. Looking at the list of options, it was felt that to achieve as much a natural appearance as possible, and give every possibility for revegetation, the following option would be the best design. In the left-hand gully, a 15-ft (4.6 m) high geotextile-reinforced rock wall (Wu, 1994) would enable a reconstruction of a 1½:1 slope above it to match the top of the cutslope. This design provided a method to "heal" the cutslope. Rock slope reconstruction techniques were similarly used with success on the Interstate 70 Glenwood Canyon Project. Amoco woven geotextile 2044 (wide width tensile strength-4800 lbs/ft. ASTM-D-4595) was used in 12-in (30 cm) compacted lifts of minus 6-in (15 cm) fill. Boulders from 3 to 6 ft (1-2 m) in diameter were used as the facing. The reconstruction would have no additional vertical exposure above the ground surface, so avalanche shear and normal stresses would not increase (Mears, 1979). Upon consultation with the CDOT Region 3 geologist and the CDOT Geotechnical Unit, this design was selected and approved.

Many revegetation-matting types are available on the market, in the form of both erosion-control matting (ECM) and turf-reinforcement matting (TRM). Most are organic-based stitched rolls of straw, coconut fiber, or shredded wood. Stronger and more expensive polypropylene, nylon, and polyester TRM are also available. Factors considered for selection of matting type at the Top Cut are altitude, harsh climate, reduced growing season, a west slope aspect, and a fairly steep, reconstructed 1½:1 slope. Also, static and dynamic snow loading had to be considered. An aggressive, very strong, soil anchorage for vegetating steep, barren slopes was required. Based in part on CDOT's experiences and testing at the Berthoud Pass Project (Goff, 1998; Zisman, pers. comm.), the IPF selected two products, Tenax Multimat and the stronger Colbond Enkamat S20. Both of these TRM materials consist of a three-dimensional (3D) geomatrix of heavy nylon monofilaments. The Enkamat "S" type is further reinforced by being thermally bonded to a strong (1560 lbs/ft, ASTM D4595) polyester geogrid. TRM is anchored to the slope by pins or j-hooks and covered with topsoil. Buried and protected from UV exposure, the matting will last many years as vegetation slowly establishes itself on the stabilized slope. The high-strength 3D TRM material can be expensive: current prices for the stronger, geo-grid bonded TRM, depending on quantity, are around $7 per square yard.

In the Fall of 1996 and 1997 the Independence Pass Foundation and CDOT completed this initial project in coordination with seasonal CDOT Maintenance ditch cleaning. The combining of projects worked well, as savings were realized by sharing mobilization costs and through use of CDOT earth moving equipment. The material generated from scaling operations of the Rockfall Control Project, and debris cleaned out of the ditch lines, was used as fill for the reinforced rock walls and the slope reconstruction. The fill derived from the disintegrated crystalline rock, minus the large rocks, served as an excellent granular material with less than 10% passing the #200 sieve. (Wright Water, Engineering, Inc., 1998)

The wall abutments were built tight against stronger rock exposures on the sides of the gullies. That, and a lack of surplus fill, prevented the construction of a ramp to reach the slope above the walls, so rocky fill above the rock walls had to be placed by crane bucket and smoothed and compacted by hand. The final slope was smoothed and prepared with topsoil donated by the Village of Snowmass and hauled by CDOT.

Installation and anchorage of the Enkamat S20 followed. The length of time that it takes for vegetation to establish itself in this location and the stresses of seasonal snow loading made it necessary to take extra care with installation. Matting anchorage was strengthened by the use of 18-in (46 cm), #4 rebar J-hooks. A 1 to 2-in (2.5 to 5 cm) layer of screened (<3-in, <7.6 cm) fines was then placed atop the TRM and worked into the 3D matrix of the matting. A seed mix designed for this altitude was broadcast over the restored slope, plants were installed, and the slope fertilized and hydromulched. The photos in Figure 5 show the before and after look of the reconstruction.

Figure 5. Left gully at Twin Gullies showing before and during slope reconstruction when TRM is being installed. Note clumps of vegetation and top soil in small debris fan at base of slope in left photo.

IPF's slope reconstruction and revegetation project realized substantial savings by utilizing inmate work crews from the Buena Vista Correctional Facility. The inmate crews of seven to eight men traveled to and from the site four days per week over a six-week period while the Twin Gullies work was underway. Besides the construction and restoration work, the inmates also spent several days planting trees and shrubs on the slopes below the road and they installed curbing along the road to channel and direct runoff. Pitkin County and CDOT Maintenance also supplied equipment and operators. IPF expenses included the woven geotextile, a crane for placement of fill and slope reconstruction material, and a local contractor hired to build the walls. Cost to IPF for the rock walls was only $5 per square face foot. The slope reconstruction was more costly because a crane was required to place the fill.

At the head of the ravine of the right Twin Gully, ten 6' by 9' by 6" thick gabion mattresses were installed and anchored to #10 threaded bars drilled and grouted into the underlying rock. 3 to 6-in (7.6 – 15.2 cm) diameter rock was screened from the fill stockpiled by CDOT and transported by crane basket to the gabion mattresses. The mattress lids were closed and wired and 6 by 6-in (15.2 x 15.2 cm) plates were bolted down on the bars sticking through the mattresses. The anchored mattresses will prevent further headward erosion of the ravine. This work was high on the slope and dangerous so inmate labor could not be used.

Erosional Amphitheater Site

In addition to the twin gullies reconstruction, other prototype work was completed. Figure 6 shows a rock-faced, geotextile reinforced slope constructed in a small erosional amphitheater in very weak decomposed rock near a large shear zone. As with the Twin Gullies reconstruction, TRM was also installed. This work was high on the slope and required a crane for mobilization of the rock and fill by bucket. The dangerous high slopes required technical rock climbing so contract labor at high-scaling rates was required. Total costs were $75 per square face foot for the wall, and $30 per square foot for slope reconstruction including TRM installation. Figure 7 is a close up photo of that reconstructed slope and shows the anchored TRM prior to being covered by topsoil.

Figure 6. Slope reconstruction is erosional amphitheater. Erosion was cutting into weak rock of a shear zone. During the 2000-2002 project, custom-colored anchored shotcrete was applied to the base of this slope below the rock wall.

Figure 7. Close-up of TRM installation at erosional amphitheater. Note geo-grid backing in the matting panels.

Stripped Vegetation Stabilization Test Sites

Work also occurred on the embankment slopes below the road in the Stripped Vegetation Zones. Test plots below the roadway were stabilized with donated ECM. Test planting terraces were also constructed and lined with matting in the stripped vegetation zone (with the highest sediment yield, see Table 1) to control further erosion. Buena Vista inmates did this work so the only cost to IPF were supervision, soil amendment, and plant material.

Big Cut Site

The success of the mechanically stabilized rock-buttressed wall for slope reconstruction led IPF to begin a new project for a larger and even more degraded cutslope at the Top Cut. This slope at milepost 59.9 is called the Big Cut. The Big Cut is the longest and highest of the eroding cut slopes along the Highway 82 Top Cut area. The cut length is 600 ft (183 m) and the highest part is 100 vertical feet (30.5 m) above the road surface. The slope cut exposure was mapped as Type 4, Decomposed Rock and Soil. This slope had also been rated in CDOT's statewide rockfall hazard rating system (Andrew, 1994). Slope grades at the site range from the standard angle of repose (34°) of the debris fans, to overhanging at the brow (scarp). The Big Cut slopes are completely barren. The Figure 3 photo was taken of the raveling slope at the Big Cut.

The height of the Big Cut slope would not permit slope reconstruction from the top of a rock wall to meet the brow of the cut slope at the 1½:1 grade. Therefore, complete revegetation of this entire slope was judged to be infeasible. Even so, slope stabilization and rockfall protection was warranted to prevent further erosion at the oversteepened tundra brow and to mitigate the rockfall danger. Wire mesh, underlain by TRM, would contain further loss of the tundra brow and eliminate rockfall at this site. To prevent further slope material loss, and to foster partial revegetation, the wire mesh will be draped on the cutslope below the anchorage, down to the top of the reconstructed slope. This rock wall and slope reconstruction was designed for staged, or phased, construction as funding and/or material became available

A formal geotextile-reinforced soil wall design was prepared by Ketchart and Wu (1998). Based on certain design criteria given them, they proposed a 21.5-ft (6.6 m) wall embedded 1.5 ft (0.5 m) below road grade, giving an effective height of 20 ft (6 m). No steeper than a 1½:1 slope would then be constructed above the wall and extend to meet the existing cut slope. The upper portion of the slope would be scaled and smoothed and draped with wire mesh. A schematic of the design is shown in Figure 8.

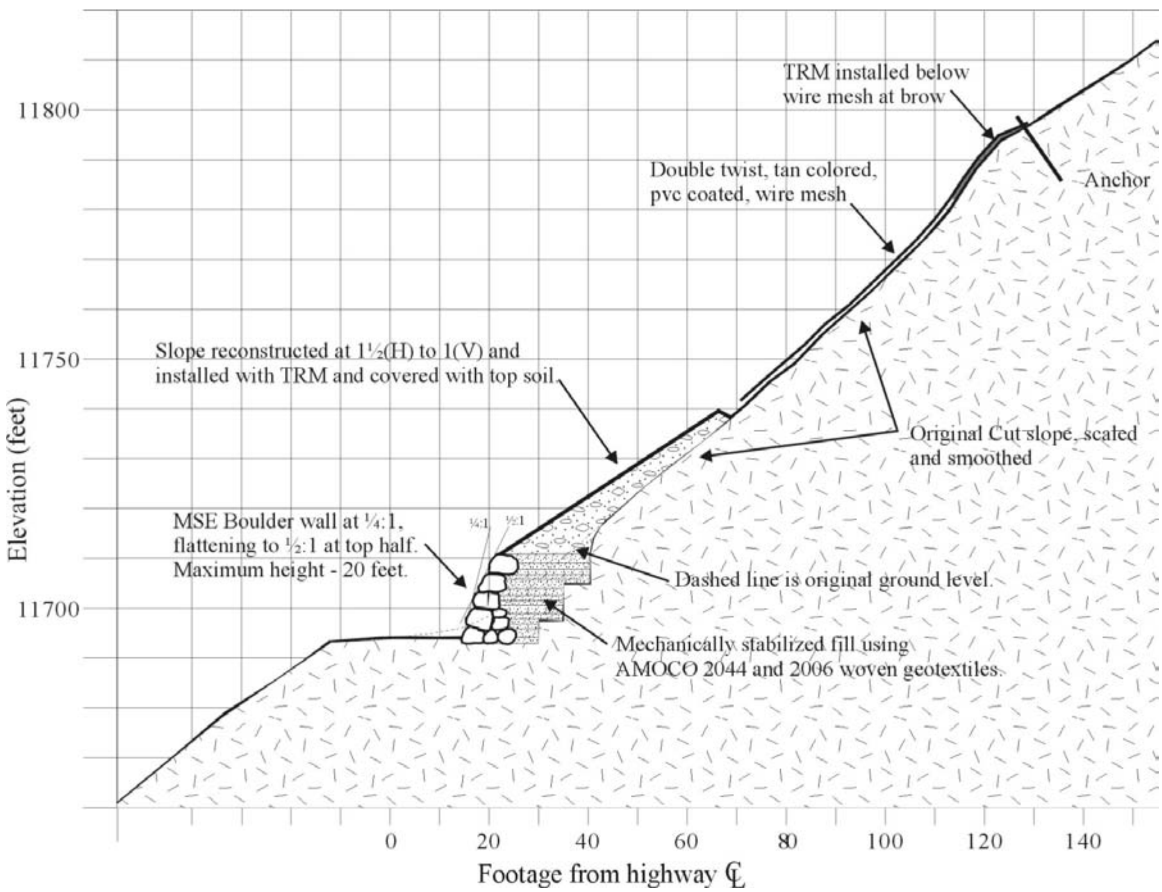

Figure 8. Schematic of slope-reconstruction design at the Big Cut.

Construction of the wall began in the fall of 1998 and work continued incrementally, as funding became available, through 2002. The foot of the cut slope was excavated back 14 ft (4.3 m) from the ditchline and boulders were placed by an excavator with a bucket thumb. As with the prototype reconstruction project, minus 6-in (15 cm) fill was placed in 12-in lifts (30 cm) and compacted. To prevent the sand and gravel fill from migrating from between the boulders, a cheap non-woven geofabric wrapped the fill at the boulder gaps. The excavator operator became highly skilled in selection of boulders, bedding, and placement to lock the boulders in place and engage the maximum friction with the reinforcement geotextile. The rolls of reinforcement are 18 ft in width and required cutting and waste for the lower lifts that were only 14 ft in width. As the wall rose and the back slope stepped back and away, the entire 18-ft width was placed in lifts, instead of the design width of 14 ft. This resulted in a much stronger wall.

The functionality of this design was very favorable for the location. The rock facing is natural rock, and while obviously a man-made wall, it is compatible with the high alpine environment. The boulders also can accommodate minor settlement with no visible or structural effects. Another design advantage is that the wall, by virtue of the gaps between the boulders and excellent granular fill, is free draining. Figure 9 is a picture of the construction of the Big Cut wall.

Figure 9. Construction of the geotextile reinforced-soil wall at the Big Cut.

CDOT 2000-2002 Rockfall Mitigation Project

In 2000 additional CDOT rockfall mitigation funds became available for the Top Cut. A combined CDOT and IPF project was developed in 2000 and 2001 that included three sites at the Top Cut. Site 1 included the Big Cut and Sites 2 and 3, farther up the Top Cut, were located at high rockfall-rated slopes. Environmentally sensitive project designs were developed that included draped custom-colored wire mesh and cable nets, scaling, rockbolts, and custom-colored application of shotcrete. Project sites are shown in the Map Plate 2.

A major portion of this combined project was the preparation of the cut slope at the Big Cut above the boulder wall. Plans included the shaping of the cutslope brow so that wire mesh would lay flat on the slope, hold TRM in place, and not create gaps under the mesh where soil could continue to ravel. The schematic for this work is shown in Figure 10. As can be seen in

Figure 4, significant shaping and scaling of the brow was needed. This was accomplished mostly by using what the contractor called a *spyderhoe*, an excavator with legs that can "walk" up 1:1 or 100% grades. Figure 11 is a picture of the spyderhoe in action at the Big Cut.

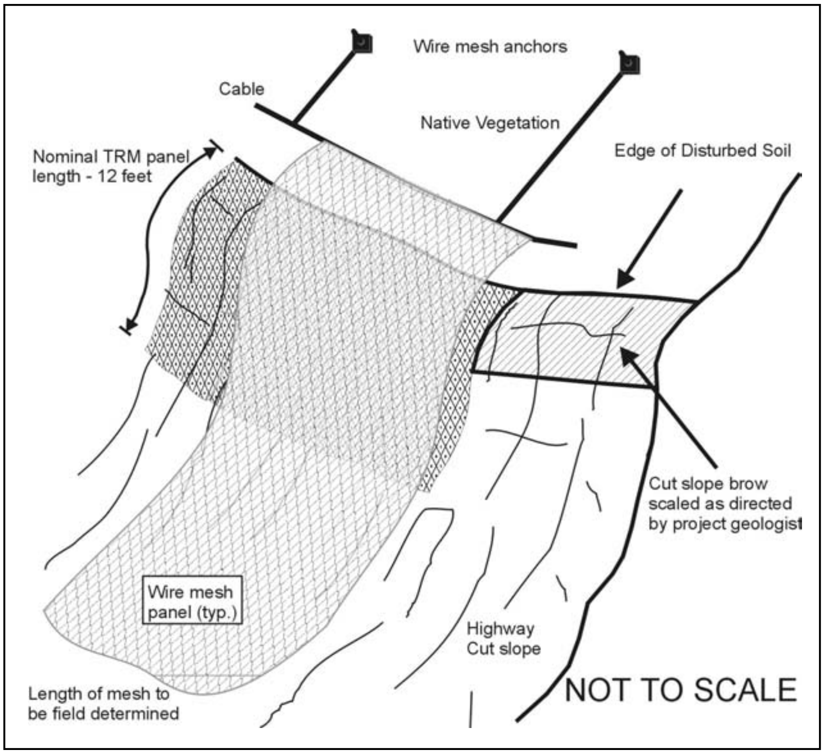

Figure 10. Schematic of cut slope brow treatment.

The project was completed in Spring 2002. By using the innovative spyderhoe and through certain value-engineering proposals accepted by CDOT, significant cost savings were realized. Savings were such that additional rockbolt locations and increased wire mesh and cable netting installations could be added to the project scope while staying under the original project budget. After the completion of the combined CDOT and IPF project in the Spring of 2002, IPF proceeded to complete the 1½:1 reconstruction of the slope above the rock wall. Rocky fill derived from scaling operations was used for the slope. The slope was smoothed and a nominal 2-3 inches (2.5 5 cm) of topsoil was placed. TCM was installed and topsoil was dumped on the slope by crane basket. Department of Corrections inmates raked the fill into the 3D matrix and covered the TRM with another 2 inches (5 cm) of topsoil. Trees were planted, the slope was seeded and fertilized, and hydromulch and tackifier were applied. Figure 12 shows slope reconstruction in progress.

Figure 11. Spyderhoe scaling 1:1 slope at Big Cut.

Figure 12. Nearly completed Big Cut Project. Wiremesh is in place on cut slope above and TRM (black material) is being staked to the reconstructed slope.

Future Work

The IPF continues to work on restoration at the Top Cut. Now that the Big Cut is complete and the CDOT rockfall mitigation project has addressed the most severe rockfall hazard sites at the Top Cut, IPF will focus restoration work in other areas of the Top Cut. For example, plantings continue on the embankment slope in the Stripped Vegetation Zone below the roadway. ECM test panels installed in the Stripped Vegetation Zone have been evaluated for 5 years now and show improved performance compared to only seeded and hydromulched slopes. A curb has been installed along the entire alignment of the road at the Top Cut to channel water flow to culverts and to prevent storm flooding and snowmelt from sheeting randomly onto the embankment. The explanation in Map Plate 2 shows the locations of the various project sites at the Top Cut. Below the road in the Stripped Vegetation Zone, ECM installation is slated for the future. The next large project is another large badly-eroded cut slope at even higher altitudes of 11,900 feet (3,627 m), called the Middle Cut. The cut slope restoration approach will be the same as the Big Cut, but fortunately the cut slope height is lower and design analysis shows that the slope reconstruction can reach the brow of the cut, creating a more elegant solution similar to what is shown in the photos of the Twin Gullies. The scar of the cut slope will be completed covered and wire mesh will not be needed.

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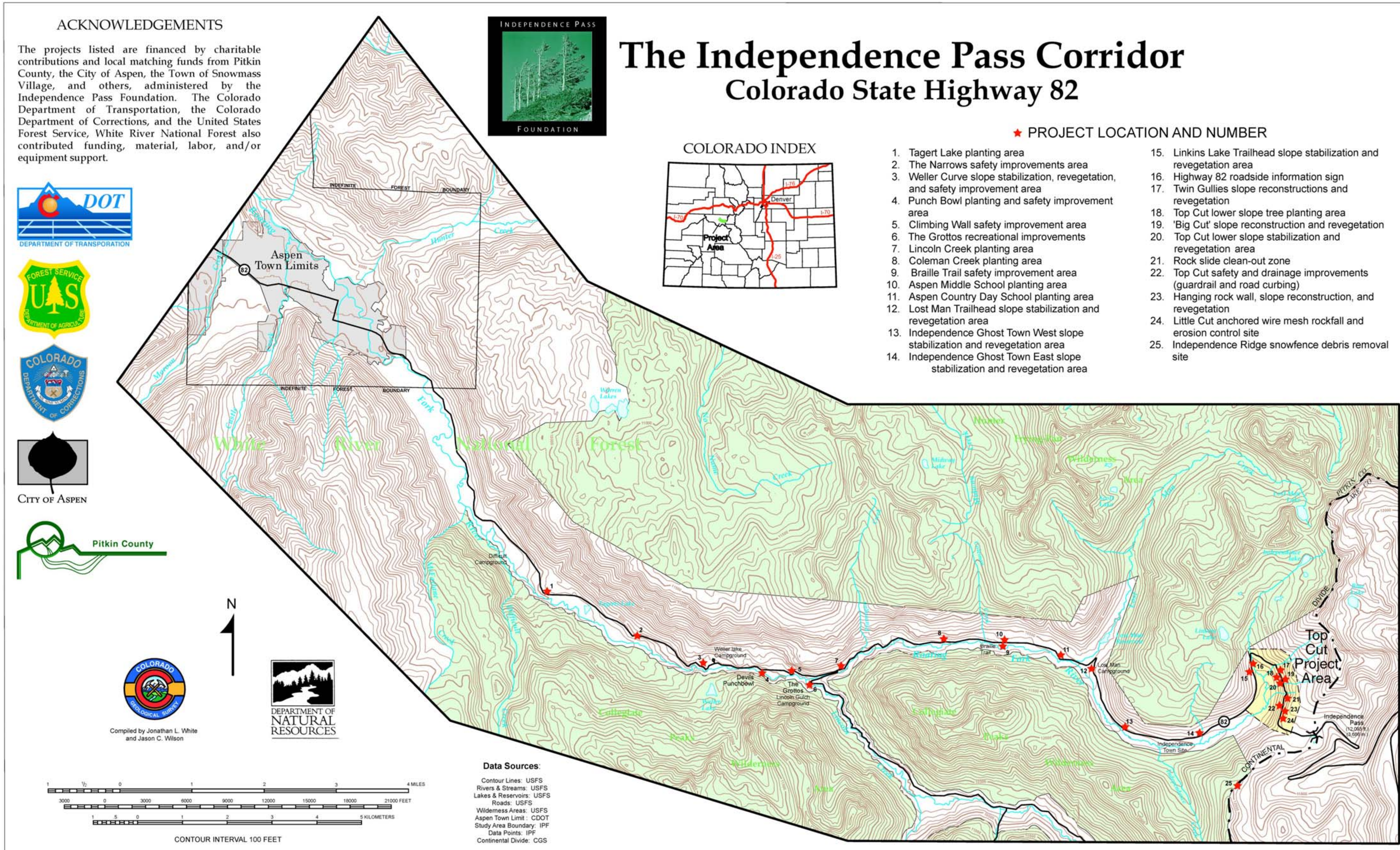
ACKNOWLEDGEMENTS
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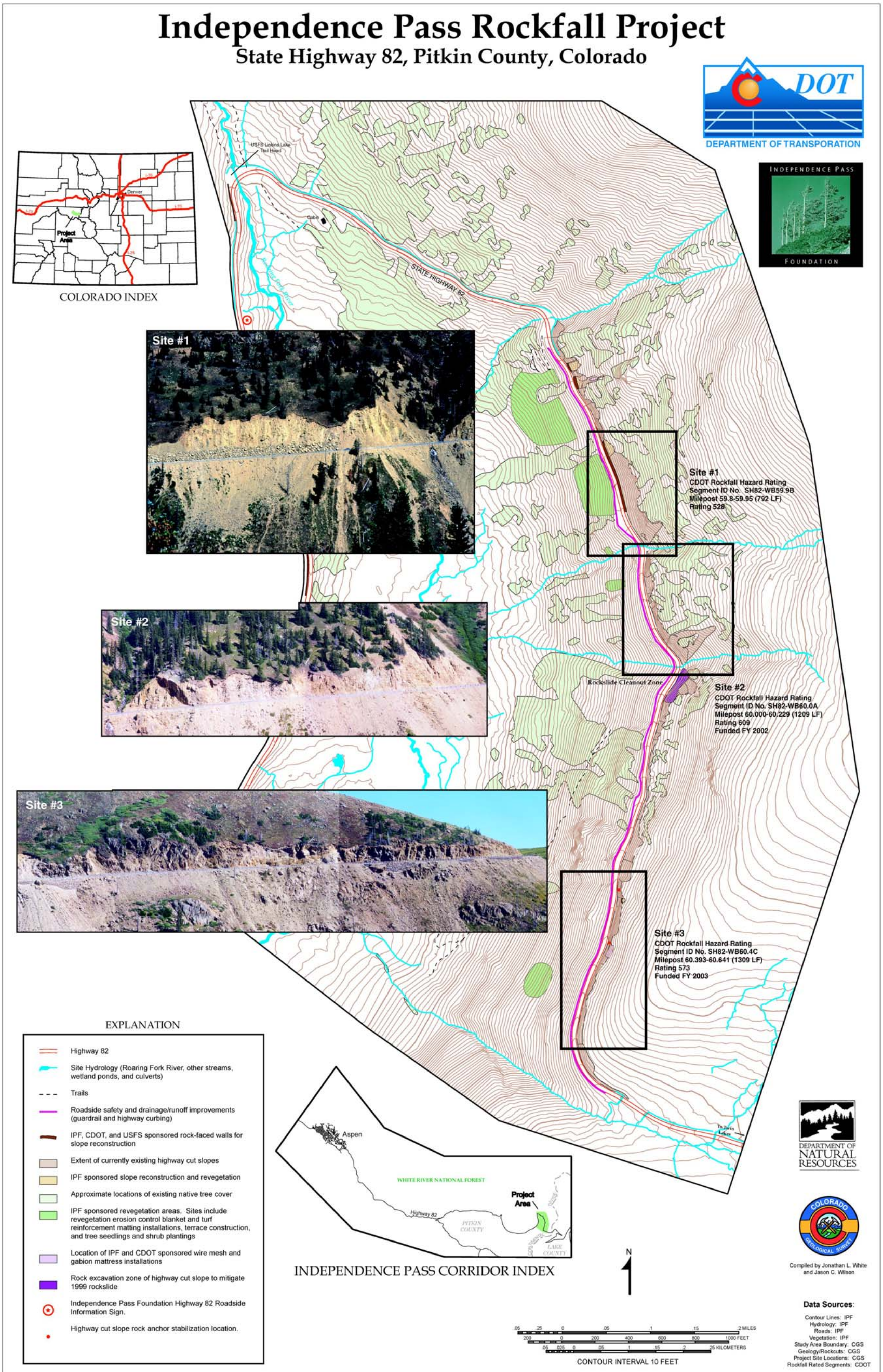
INDEPENDENCE PASS
FOUNDATION
The Independence Pass Corridor
Colorado State Highway 82
COLORADO INDEX
Denver
Project Area
PROJECT LOCATION AND NUMBER
1. Tagert Lake planting area
2. The Narrows safety improvements area
3. Weller Curve slope stabilization, revegetation, and safety improvement area
4. Punch Bowl planting and safety improvement area
5. Climbing Wall safety improvement area
6. The Grottos recreational improvements
7. Lincoln Creek planting area
8. Coleman Creek planting area
9. Braille Trail safety improvement area
10. Aspen Middle School planting area
11. Aspen Country Day School planting area
12. Lost Man Trailhead slope stabilization and revegetation area
13. Independence Ghost Town West slope stabilization and revegetation area
14. Independence Ghost Town East slope stabilization and revegetation area
15. Linkins Lake Trailhead slope stabilization and revegetation area
16. Highway 82 roadside information sign
17. Twin Gullies slope reconstructions and revegetation
18. Top Cut lower slope tree planting area
19. 'Big Cut' slope reconstruction and revegetation
20. Top Cut lower slope stabilization and revegetation area
21. Rock slide clean-out zone
22. Top Cut safety and drainage improvements (guardrail and road curbing)
23. Hanging rock wall, slope reconstruction, and revegetation
24. Little Cut anchored wire mesh rockfall and erosion control site
25. Independence Ridge snowfence debris removal site
DOT
DEPARTMENT OF TRANSPORTATION
FOREST SERVICE
DEPARTMENT OF AGRICULTURE
COLORADO DEPARTMENT OF CORRECTIONS
CITY OF ASPEN
Pitkin County
Aspen Town Limits
INDEFINITE FOREST BOUNDARY
White River National Forest
Top Cut Project Area
Independence Pass
CONTINENTAL DIVIDE
N
Compiled by Jonathan L. White and Jason C. Wilson
COLORADO GEOLOGICAL SURVEY
DEPARTMENT OF NATURAL RESOURCES
CONTOUR INTERVAL 100 FEET
Data Sources:
Contour Lines: USFS
Rivers & Streams: USFS
Lakes & Reservoirs: USFS
Roads: USFS
Wilderness Areas: USFS
Aspen Town Limit: CDOT
Study Area Boundary: IPF
Data Points: IPF
Continental Divide: CGS

Map Plate 1



Independence Pass Rockfall Project
State Highway 82, Pitkin County, Colorado
DOT
DEPARTMENT OF TRANSPORATION
INDEPENDENCE PASS
FOUNDATION
COLORADO INDEX
Denver
Project Area
USFS Linkins Lake Trail Head
Cabin
STATE HIGHWAY 82
Site #1
Site #2
Site #3
Site #1
CDOT Rockfall Hazard Rating
Segment ID No. SH82-WB59.9B
Milepost 59.8-59.95 (792 LF)
Rating 528
Rockslide Cleanout Zone
Site #2
CDOT Rockfall Hazard Rating
Segment ID No. SH82-WB60.0A
Milepost 60.000-60.229 (1209 LF)
Rating 609
Funded FY 2002
Site #3
CDOT Rockfall Hazard Rating
Segment ID No. SH82-WB60.4C
Milepost 60.393-60.641 (1309 LF)
Rating 573
Funded FY 2003
EXPLANATION
Highway 82
Site Hydrology (Roaring Fork River, other streams, wetland ponds, and culverts)
Trails
Roadside safety and drainage/runoff improvements (guardrail and highway curbing)
IPF, CDOT, and USFS sponsored rock-faced walls for slope reconstruction
Extent of currently existing highway cut slopes
IPF sponsored slope reconstruction and revegetation
Approximate locations of existing native tree cover
IPF sponsored revegetation areas. Sites include revegetation erosion control blanket and turf reinforcement matting installations, terrace construction, and tree seedlings and shrub plantings
Location of IPF and CDOT sponsored wire mesh and gabion mattress installations
Rock excavation zone of highway cut slope to mitigate 1999 rockslide
Independence Pass Foundation Highway 82 Roadside Information Sign.
Highway cut slope rock anchor stabilization location.
Aspen
WHITE RIVER NATIONAL FOREST
Project Area
Highway 82
PITKIN COUNTY
LAKE COUNTY
INDEPENDENCE PASS CORRIDOR INDEX
N
MILES
FEET
KILOMETERS
CONTOUR INTERVAL 10 FEET
DEPARTMENT OF NATURAL RESOURCES
COLORADO GEOLOGICAL SURVEY
Compiled by Jonathan L. White and Jason C. Wilson
Data Sources:
Contour Lines: IPF
Hydrology: IPF
Roads: IPF
Vegetation: IPF
Study Area Boundary: CGS
Geology/Rockcuts: CGS
Project Site Locations: CGS
Rockfall Rated Segments: CDOT

Map Plate 2.

ENGINEERING GEOLOGY FOR RELOCATION OF A HIGHWAY IN GLACIATED TERRAIN, CLIMAX MINE AREA, SUMMIT COUNTY, COLORADO

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Jerome B. Hansen
Great Basin Exploration Consultants, Inc., 143 Union Blvd., Ste 230, Lakewood, CO 80228

Key terms: landslide, slump, glacial till, perched water table, mine subsidence, slope stability, glacial moraine, highway alignment

ABSTRACT

During 1976, the proposed increase of the tailings from Amax Corporation's Climax molybdenum mine necessitated relocation of a 4.17-mi (6.7 km) stretch of State Highway 91 north of Fremont pass in Summit County, Colorado. The geologic conditions along the proposed alignment from Sta. 35+00 to the northern terminus (Sta. 220+00) were mapped, described, and evaluated by geologists from Amuedo and Ivey under contract to Centennial Engineering, Inc., the design engineers for the project. This paper summarizes the findings of the study and the recommendations made to that firm.

Potential geologic hazards and problem materials were identified for the design engineer. These included landslide and slump features, perched water tables, areas of excessive runoff, and mine-related subsidence. Problem materials include areas of unstable glacial till, fault zones, talus trains, and oversteepened slopes. The nature of the geologic conditions was such that engineers would be pressed to make field decisions on the treatment of problems, many of which would only be found after construction was under way.

Construction began in the spring of 1977 and the highway was opened in September 1978. On July 31, 1982 an informal inspection was made and only minor failures were observed. Several other trips along this alignment since then have been made without any indication of major failures.

DESCRIPTION OF PROJECT AREA

The project area is located along the eastern slope of the valley of Tenmile Creek between Clinton Creek on the south and Graveline Gulch on the north (Figure 1). Elevations range from just above 10,200 ft (3109 m) to just below 11,500 ft (3505 m).

The Tenmile Creek Valley trends roughly northeastward through mountainous terrain. Most of the valley is comprised of fairly steep walls produced by glacial erosion and deposition. The

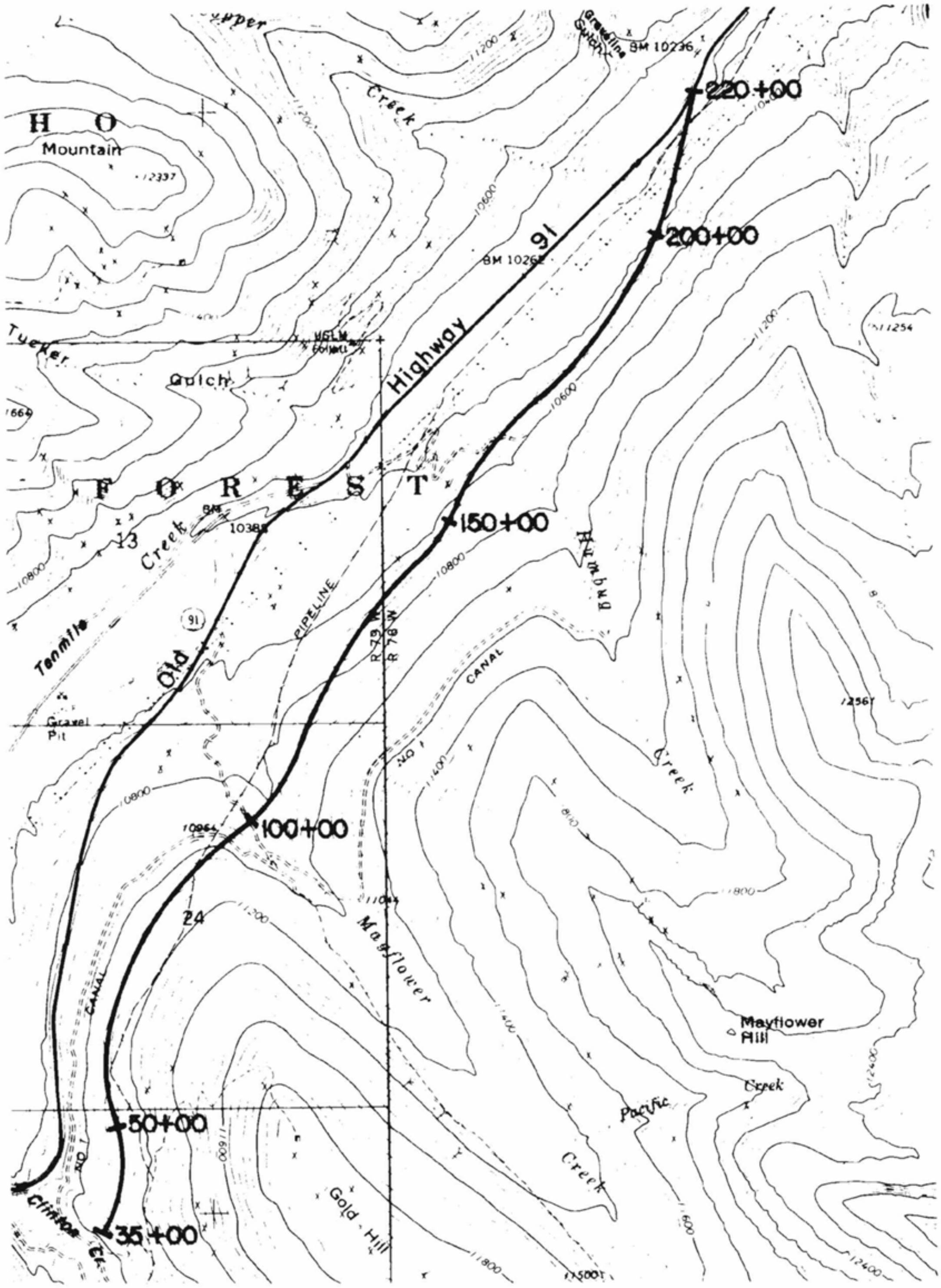

Figure 1. Highway 91 Relocation

surface of the area consists mainly of Pleistocene glacial till and outwash, and shallow Quaternary colluvium, which overlie bedrock slopes.

The major streams in the study area, Clinton Creek (now dammed), Mayflower Creek, and Humbug Creek, drain into Tenmile Creek, which flows northeastward into Dillon Reservoir, approximately 12 mi (19 km) from the project area. These streams generally run close to bank-full in the late spring due to the large quantity of melt water derived from the winter snow accumulation, which is in excess of 100 in (254 cm) in a normal season. Many small, steep-graded intermittent streams also drain the valley sides.

In addition to these streams, the alignment is dotted with many small springs and seeps. Some of these flow throughout the summer but most of them appear to slow or stop by late summer and early fall, the driest parts of the year. Many small, boggy and marshy areas are associated with the seeps, springs, and small ephemeral streams.

Before construction began, most of the study area was densely wooded with mature stands of lodgepole pine. Wet, poorly drained areas supported thickets of willows and lush, broad-leafed ground cover. Some drier areas had developed into mountain meadows covered with grasses and shrubs.

Pre-construction access to the area was by means of unimproved dirt roads and construction trails, situated along Clinton, Mayflower, and Humbug Creeks and the No. 1 canal (Figure 1). Each of these roads was accessible via the old Highway 91.

PURPOSE AND SCOPE

The purpose of this project was to collect the geologic information necessary for the engineering design of the highway, for slope recommendations, and for mitigation measures in areas of existing or potential land failure.

Several alignments had been proposed for the highway at various times. The first one was established by Amax Corporation and was staked in 1973. The Colorado Division of Highways developed a different alignment and surveyed 100 ft (30 m) stations along it in 1974 and 1975. Profiles for these stations were developed from topographic maps produced in 1968. Surveyed elevations were established at random intervals along the centerline of the alignment staked by the Division of Highways.

The scope of the work involved a review of data previously assembled by Climax Molybdenum Company and its consultants, and by the Colorado Department of Highways. This data included surficial and bedrock mapping, seismic surveying, borehole inclinometer data, and information acquired from prospect pits. Since most of these data had not been prepared specifically for the highway project, it was necessary to do engineering geologic mapping, additional geophysical work, test pit excavation, and test drilling. The evaluation of data from all sources provided the input to the design engineers.

PROJECT CONSTRAINTS

This project was unique, in that options for alternate alignments were restricted within a few feet in the most critical areas. The alternate alignments that had been staked were within a few feet of each other. Because alignment options were so severely restricted in an engineering sense, it was apparent that the geology would be a major control on the project. Hence, the engineering geologic investigations concentrated on a field review of known problem areas, and the identification of previously unrecognized ones.

Poorly exposed geology made it impossible to determine precisely in a cursory review where failure and distress in natural and manmade features could be expected. Certain areas were identified from direct observation as being prone to failure, and it was apparent that other areas would be identified during construction. It was also evident that the extent to which geologic conditions were masked by vegetation and surficial material would force many engineering decisions to be made in the field during construction.

METHODS AND PROCEDURES

An office photogeologic review of the area was made on aerial photographs and detailed field mapping of the project was conducted between July 28 and September 21, 1976. Mapping was done on topographic bases at a scale of 1 in = 200 ft, supplied by Climax Molybdenum Co., and annotated with the known regional geologic data. A control network along the alignment was surveyed by Climax in the most densely vegetated areas to provide reference points for detailed geologic mapping by Brunton compass and tape/pace traverses, and by triangulation methods.

Specifications were developed for four test holes, which were drilled by the Colorado Department of Highways under contract to Climax. Three holes were drilled along a line normal to the projected alignment on centerline and to either side, just south of Mayflower Creek, where bedrock was shallow. A fourth hole was drilled in the alluvium of the creek on centerline. Samples were collected of the unconsolidated materials in each hole at irregular intervals, and bedrock was cored to a few feet below grade. All samples were described and cores were logged for lithology, structure, and Rock Quality Designation (RQD, Deere et al, 1969).

A test pit program was designed and executed jointly by Amuedo and Ivey and R. W. Thompson, Inc., the project soils engineering consultant, to test surficial material and measure the depth to bedrock. Eleven pits were dug with a backhoe, descriptions were made, and samples were collected for testing by Thompson. This work supplemented a test pit program that had previously been conducted (Amax, 1975a).

GENERAL GEOLOGY

The geology of this area has been of interest since the late 1800's because of the potential for economic mineral deposits. The Climax molybdenum mine about four miles south of the new alignment is the most important mining operation in the area. The general geology of the area is shown in Figure 2.

Stratigraphy

The major bedrock units in the area consist of the Late Paleozoic sedimentary Minturn and Maroon Formations, which overlie Precambrian metamorphic gneisses and migmatites. Several Tertiary intrusives have invaded both the sedimentary and metamorphic rocks of the area. All bedrock units are overlain by glacial till or post-glacial unconsolidated deposits ranging in thickness up to 100 ft (30 m).

The Precambrian metamorphic rocks are highly altered metasedimentary gneisses and migmatites of the Idaho Springs Formation. The principal minerals in these rocks are quartz, feldspar, biotite, and hornblende. Feldspar crystals are preferentially aligned and clusters of quartz crystals are readily observed in the rock. Banding in the gneisses shows evidence of both flowage and failure (Bergendahl, 1963, Bergendahl and Koschmann, 1971). Pronounced foliations are easily observable in some of the migmatites and essentially absent in others.

The Minturn and Maroon Formations are characterized by red-to-gray arkoses, siltstones, and mudstones (Bergendahl, 1963). Many of these rocks contain abundant mica, often in layers along bedding planes. The most extensive exposures of these rocks were found in the deep roadcut just south of Clinton Dam. Intensively metamorphosed and mineralized quartzites and arkoses are locally associated with the sedimentary rocks. Due to poor exposures, the relationship of these rocks to the Minturn and Maroon Formations could not be precisely determined.

The Tertiary igneous intrusives in the area generally are quartz monzonite porphyries and the only variations between them are in the accessory minerals and grain sizes of each (Bergendahl, 1963). Some poorly developed contact metamorphism was suspected in the host rocks and adjacent to the intrusives.

Morainal material and associated outwash and alluvial fans overlie the bedrock along Tenmile Creek and all of its major tributary streams: Clinton, Mayflower, Humbug, and Copper Creeks. Where these deposits are well drained and have a stabilizing vegetation matte, they appear to be relatively stable. Where exposed to excessive moisture, these deposits locally become highly mobile and unstable at existing slope angles.

Above the major valley floors, glacial material interfingers with slope wash, talus, colluvium, and residuum. Much of this material is also unstable at the surface, and many small-scale landslides, slumps, and stepped surfaces have developed.

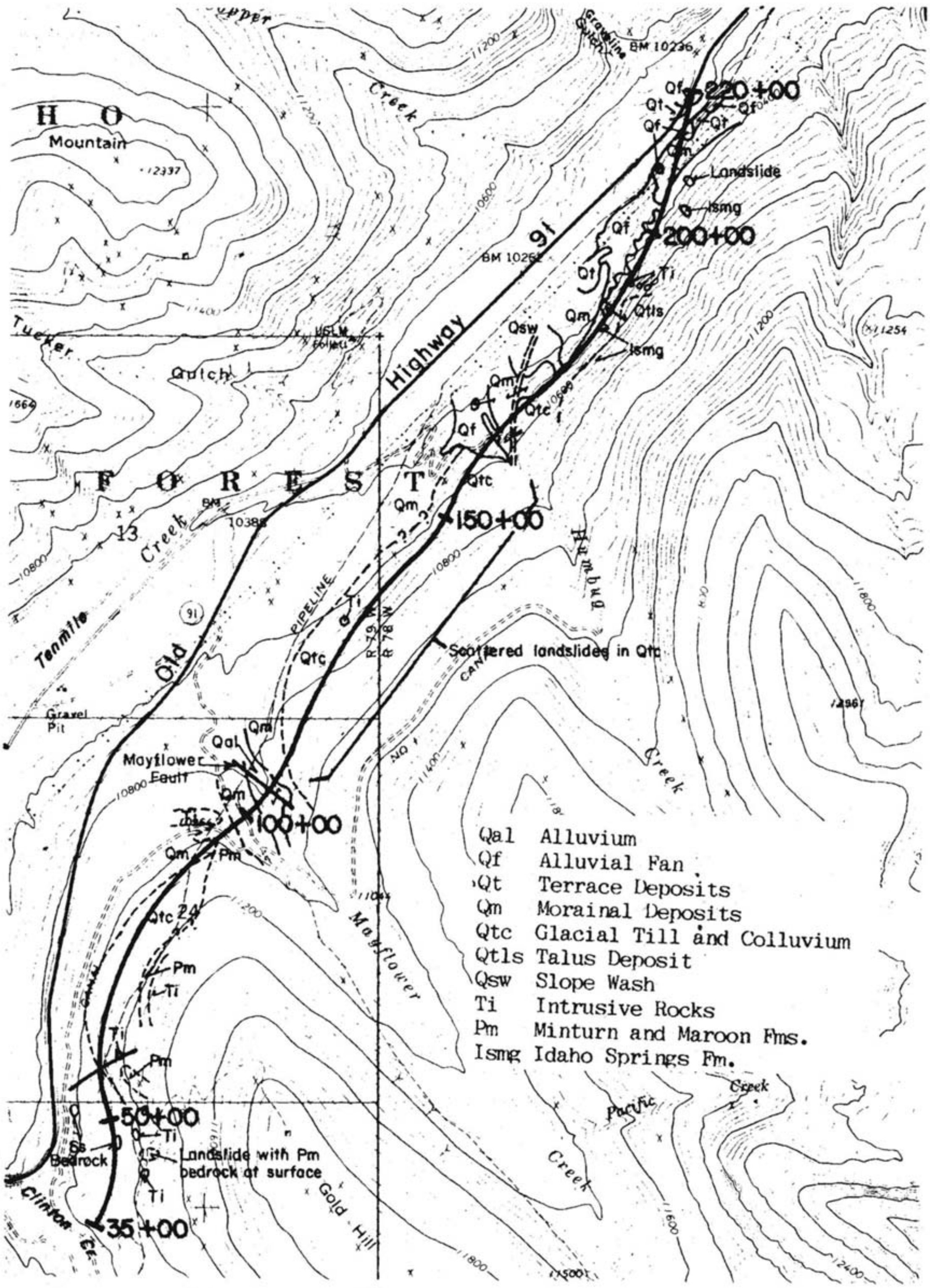

Figure 2. General Geology

Structure

The sedimentary and metasedimentary units in the area have dips ranging from 25-70º to the west and northwest in the few locations where they could be measured. Mapping of the area (Amax, 1975b) has shown that major faults intercept the alignment in three places. Surface evidence of this faulting was not observed during the detailed mapping program in the narrow strip studied.

The bedrock exposed in the study area was extensively jointed and fractured. The limited rock exposures and the small number of data points available precluded any meaningful analysis of the relationship between the joints and fractures observed and the regional structural patterns. Much of the fracturing may be due to relatively recent ice action and may not persist to significant depths.

ENGINEERING GEOLOGY

The engineering geology investigation included determination of depth to bedrock, engineering properties of the rocks and unconsolidated sediments, hydrology, slope stability, and the consideration of problem areas. The physical characteristics and stability of unconsolidated materials were quantified by the project soils engineering consultant.

The generally poor and limited exposures of bedrock along the alignment made impossible the direct observation of structural features, which control cut-slope stability. A zone approximately 1000 ft (305 m) wide or 500 ft (152 m) on either side of the alignment centerline was mapped, first on aerial photographs and then on the ground.

Depth to Bedrock

Depth to bedrock at various locations throughout the alignment was determined by drill holes, backhoe test pits, rock outcrop projection, and seismic surveys. Overburden thickness varies from zero to over 100 ft (30 m). Most of the alignment is within areas underlain by glacial till and colluvium (Figure 2). The alignment crosses thick deposits of glacial moraine and outwash where Clinton, Mayflower, and Humbug Creeks enter the valley of Tenmile Creek. Significant zones of shallow or exposed bedrock occurred within the mapped area generally about 250-to-500 ft (76 to 152 m) east or upslope from the centerline. An exception to this was on the south side of Mayflower Creek where rock exposures were mapped on both sides of the centerline.

The depth to bedrock, and the configuration of the bedrock-overburden interface were of particular concern in the so-called "tenderloin" area. There, the depth to and configuration of the bedrock surface was estimated from surface mapping, drill-hole data, seismic investigations, and test pit excavations.

The bedrock exposure was controlled largely by the glaciation of the valley of Tenmile Creek. The northward-moving ice left a relatively smooth bedrock wall that slopes northwestward on the eastern side of the valley. On this wall lies a vegetated cover of glacial material, weathered rock, and colluvium of variable thickness and distribution. Irregularities in the bedrock are found locally, and these are generally oriented parallel to or nearly parallel to the valley. Local bedrock highs may have formed buttresses against the downslope movement of unconsolidated overburden.

The overburden thins to the southeast, uphill from the alignment. This thinning was observed in prospect and assessment pits. Generally within 1000 ft (305 m) east of the alignment the overburden is from 5-10 ft (1.5-3 m) thick.

Engineering Properties of Geologic Units

Sedimentary Rocks – The Minturn and Maroon Formations consist primarily of sandstones, siltstones, and mudstones, which, if undisturbed, provide a good sub-base for the roadway. These rocks appeared to be well jointed in the few exposures nearest the alignment (Figure 3). Alteration of these rocks by weathering probably has produced zones of weakness. Mechanical weathering likely produced small fragments, which subsequently acted as unconsolidated material. Chemical weathering and alteration produced zones or layers of plastic, easily deformed and easily eroded material. The presence of mica in these rocks may have contributed to their instability in local areas.

An initial concern with bedded sedimentary rocks was instability in cut slopes where bedding or joint-planes would be undercut. Unfortunately the exposures of these rocks were limited in extent and were too far from the alignment to be of value for any definitive analysis in the areas of interest. The high degree of fracturing observed in the few exposures along the alignment and in the test core drilling implied that most of the sedimentary bedrock encountered at or above final grade would be rippable.

Igneous and Metamorphic Rocks – The igneous intrusives of Tertiary age in the study area are quartz monzonite porphyries that are relatively sound and stable in their unaltered state (Figure 4). If these rocks are encountered in a weathered and (or) fractured condition at grade along the alignment they could cause stability problems.

The metasedimentary rocks of Precambrian age observed in the study area are primarily gneisses and migmatites. One exception is a fairly common metaquartzite associated with the sedimentary units described earlier. The gneisses and migmatites displayed poor- to well-developed foliation. These foliations represent zones of weakness along which movement or failure could occur under load or where undercut in excavations. The metaquartzite appears to be highly fractured and jointed. It was estimated that it might need to be stabilized in cuts but should provide sound sub-base beneath fills.

Exposures of intrusive rocks in outcrops or test pits along the alignment were dug easily with a backhoe because they were weathered and well jointed (Figures 5 and 6). It was determined that these rocks should prove to be rippable close to the surface where frost action and weathering have weakened them.

Figure 3. Exposure of Minturn-Maroon Formations in west side of road cut on south side of Clinton Dam.

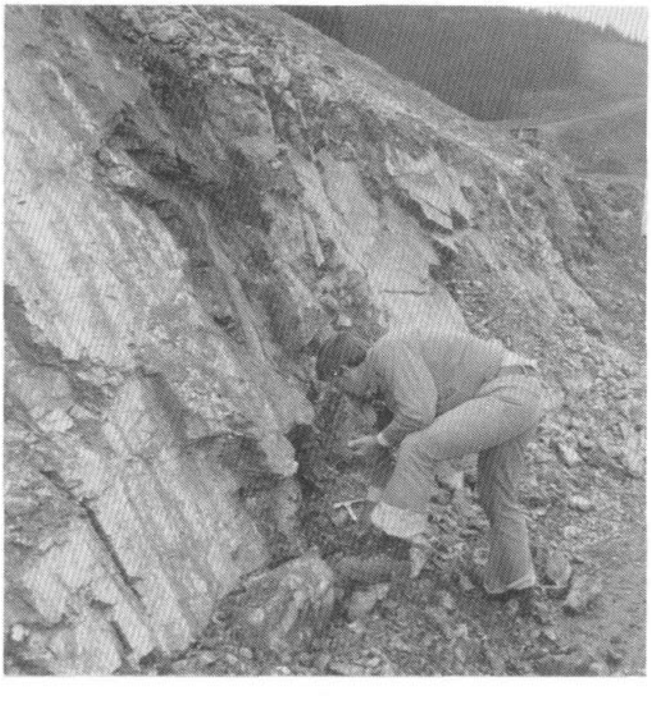

Figure 4. Exposure of Tertiary intrusive rock in trench dug on south side of Mayflower Gulch.

Figure 5. Backhoe in operation on pioneer road in “tenderloin” area.

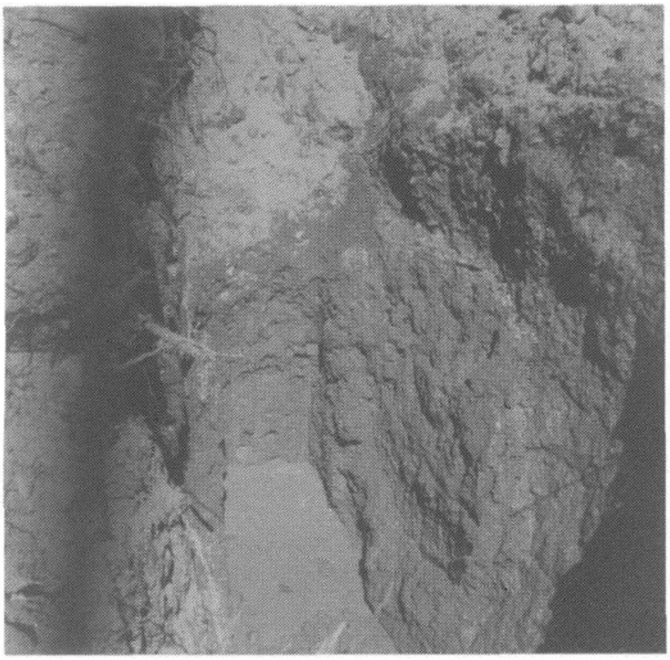

Figure 6. Backhoe pit in jointed and weathered Tertiary intrusive rock, “tenderloin” area.

Seepage zones associated with metasedimentary rocks in test pits and where rock was close to the surface indicated that, if highly jointed, metaquartzite could provide a permeable zone or channel which might cause water-related problems such as piping or slumping. It was estimated that the highly fractured metaquartzites should also be rippable.

Unconsolidated Surficial Deposits - Unconsolidated surficial deposits along the alignment consist of glacial till and outwash, colluvium, talus, residuum, alluvium, and landslide and slump material. All of these deposits can be moved with bulldozers, scrapers, and backhoes, although some boulders may be too large for equipment other than bulldozers to handle.

Till, Outwash, Colluvium, Talus, and Residuum – Glacial till, colluvium, talus, and residuum occur as an irregular mantle of unconsolidated material overlying bedrock throughout most of the area. Till also occurs in morphologically distinct landforms similar to moraines. This material is poorly sorted and unstratified (Figures 7 and 8). The mineral composition and grain size distribution vary considerably. Much till, colluvium, and talus was found in naturally oversteepened environments and may have been temporarily stabilized under certain critical conditions of moisture and (or) vegetation.

In moist, poorly drained areas, such as the “tenderloin” (Stas. 115+00 to 157+00), disruption of the surface material in any way would likely cause failure (Figures 9 and 10). Engineered drainage of slopes appeared to be the most readily applied corrective measures in this situation. Additional engineering design (i.e. retaining walls and armored slopes) was suggested in specific locations. Removal and replacement of some of this material was suggested as a remedial measure if zones of high clay content or organic matter were encountered during construction.

It was anticipated that a number of talus trains, which were buried under several feet of organic material, would be uncovered during construction. The greatest likelihood for this was in the general area north of the “tenderloin” to Sta. 185+00.

Areas of outwash and alluvium presented few problems in either construction or maintenance. These deposits are fairly permeable and could be easily excavated if necessary. Because of their mode of deposition, these materials were well compacted and should provide good foundation for fill with little or no settling except where local lenses of organic matter or clay occur. Neither of these undesirable materials was observed during the field mapping or in the test drilling conducted in Mayflower Creek. When found in sufficient quantities at the surface or above grade, organic material was stripped and stockpiled to be used for revegetation.

Residuum consists of soil and decomposed rock weathered in place. This material interfingers with the till and colluvium and is more predominant where the glacial deposits thin in an uphill direction. The residuum is generally thin, granular and contains a large amount of organic matter, silt and clay. Some of this material is very clay-rich and plastic, although “fat” clays were not observed.

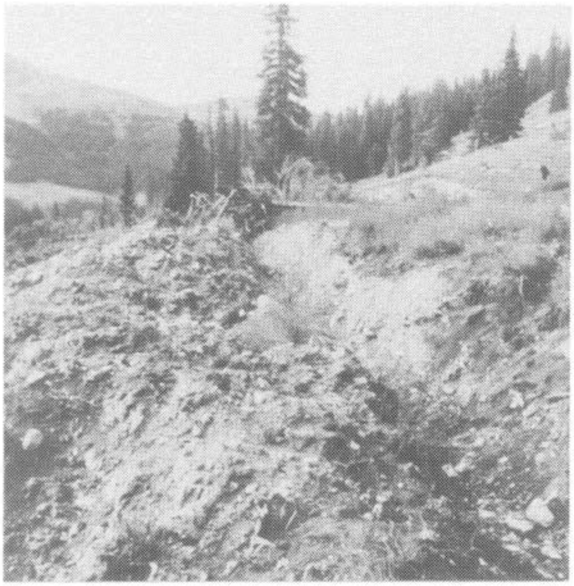

Figure 7. Glacial till exposed above Borrow Area B. Material on left (west) has slumped; note light-colored scar marking uphill limit of slump.

Figure 8. Glacial till exposed in old drill site road at about Sta. 164+50.

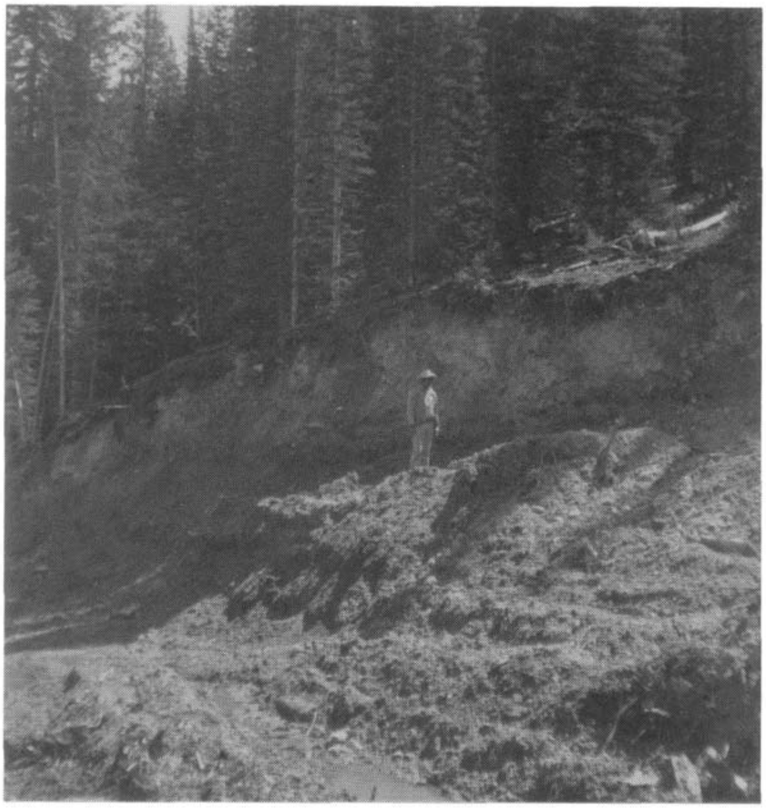

Figure 9. Slump in cut for drill road at about Sta. 124+50. After cut was made slumping apparently occurred in several episodes.

Figure 10. Small landslide or slump that occurred between August 6 and 8,1976, in morainal material at about Sta. 110 on the north side of Mayflower Creek.

Landslide and Slump – Landslide and slump materials occurred in several places along the alignment. Locally these materials were estimated to be in temporary equilibrium and extremely sensitive to disruption. These landslides may be reactivated as a result of construction activities. Most of the failure features observed during the detailed mapping were assumed to be due to small, shallow-seated shear planes. The most likely time for failure to occur is during the high-moisture period in late spring when thaw penetrates to the frost line. Figure 10 is a picture of a failure that occurred during the fieldwork as a result of heavy rains between August 6 and 10, 1976. It was estimated that much of this type of material would have to be removed or drained and reinforced to insure the integrity and safety of the roadway.

In the "tenderloin" area, soft ground was produced as a result of disruption of wet unconsolidated material. This resulted in potential hazards to heavy equipment during construction of pioneer roads and during the test pit program. Similar conditions existed in the large boggy area around Sta.165+00.

Slope Stability

Engineering geology factors regarding slope stability are described here in qualitative terms. Quantitative aspects of slope stability are covered in a soils engineering study of the site by R.W. Thompson, Inc. (Holmquist, 1983).

Bedrock – Slope stability in the highly jointed and fractured sandstones, siltstones, and the metaquartzite was determined by the strike and dip of rock encountered during excavation.

In metamorphic basement rocks, the orientation of joints and foliations determined the relative cut stability at various slope angles. Cut-slope stability in the unaltered intrusive rocks is controlled largely by the orientation and dip of joints. Generally, these rocks should be sound and should be stable in fairly steep cuts. The presence of intrusive rock also adds an element of strength to the rocks that it has intruded.

Unconsolidated Deposits – Many small cut slopes, generally less than six feet (two meters) in unconsolidated deposits were observed in the project area and appeared to be relatively stable even at near-vertical angles, whereas larger cuts appeared to be unstable even at fairly shallow slopes of about 2:1. Cut slopes much shallower than 2:1 were not practical on many parts of the alignment because they would not "daylight" for several hundred feet. Unconsolidated deposits were particularly subject to erosion and to mass failure.

Hydrology

Annual precipitation in this area is approximately 20 in (51 cm) and the majority of it falls as snow in the winter months (NOAA, 1975). A great deal of this moisture runs off during the late spring while the ground is still frozen and impermeable. The remainder evaporates, and percolates downhill in the shallow subsurface as vadose water passing through the unconsolidated material along the steep slopes of the valley.

Much of this vadose water appears to stay in the unconsolidated materials until 1) it reaches a stream course where it becomes surface water, 2) it is intercepted by small surface depressions, or 3) is encountered impermeable barriers and feeds bogs, springs, and seeps. Water reaching the surface in or just above cut slopes, or under fill sections, can cause serious problems. A preliminary site-specific drainage program was developed for the "tenderloin" by the design engineers with geological inputs. This drainage program was partially implemented in the fall of 1976 in an effort to dewater some problem areas and to facilitate construction in the spring of 1977.

Based on the field information acquired it was concluded that the water table is well below the ground surface, but that perched water tables could be expected throughout the area.

Problem Areas

Problem areas were identified in order to define conditions that should be mitigated in design (Figure 11). It should be noted that even slight changes in the proposed alignment altered the relative effect of these problems on the roadway, eliminating certain site-specific problems and introducing others not noted in this study.

Borrow Area B – This borrow area was developed in a thick wedge of glacial till northeast of Clinton Creek from Sta. 35+00 to Sta. 54+00. This deep cut had been made to provide fill for the Clinton Creek Dam prior to this investigation.

The higher slope on the right of the alignment underwent failure since it was first opened in the fall of 1974, and was still failing in September 1976 (Figure 12). Failure ranged from incipient to advanced during the period of field observation (July 28 through September 21, 1976). The most important factor associated with this problem appeared to be the extreme instability of the clay-rich unconsolidated material in the presence of excess moisture. Figure 13 is a generalized plan of Borrow Area B as of early September 1976. Note the common association of water with slumped areas.

Even as the expected period of low groundwater flow approached in the late summer, there were still many moist areas and several active springs and seeps visible in and just above the cut. The most significant of these springs was a network that issued from a slump mass possibly associated with an old collapsed mine adit above the cut, between Stas. 42+00 and 43+50. This water drained into an open, unlined trench, which ran northward parallel to the top of the cut for 140 ft (43 m) and then down the face of the cut to a plastic drainpipe. The unlined portion of the surface drainage system and ruptures in the plastic pipe operated as a recharge system to the cut area.

A lined cut-off trench was recommended along the abandoned gas-line bench at the top of this cut to divert water southward from about Sta. 45+00 into the impoundment behind Clinton Dam.

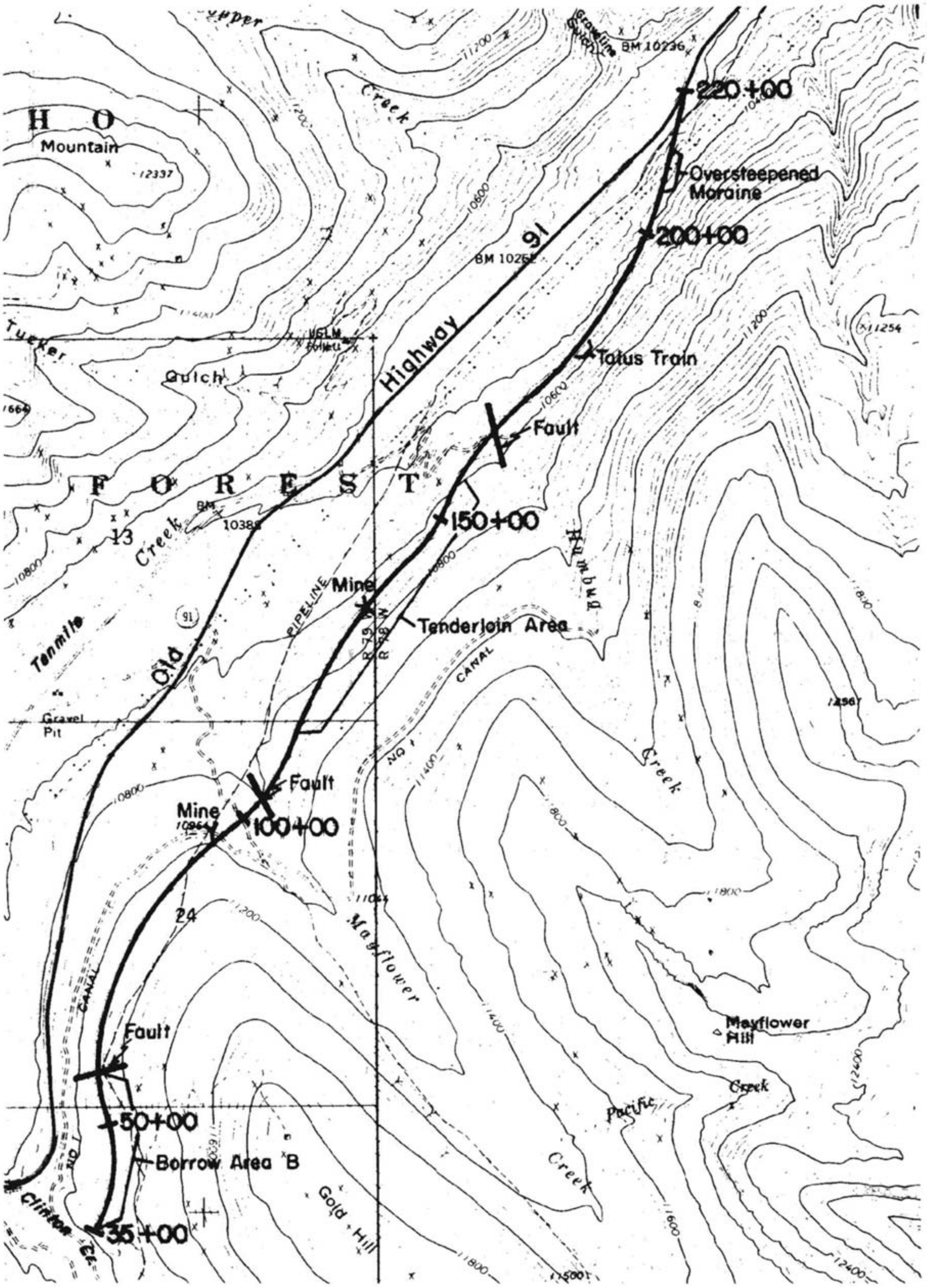

Figure 11. Problem Areas.

Figure 12. Right cut slope at about Sta. 39+00 in Borrow Area B, September 1976. Note failures indicated by multiple scarps. This picture is typical of conditions in the cut slope between Stas. 36+50 and 41+00.

Northward, from about Sta. 45+00 to Sta. 53+00, this trench would protect that part of the cut, which showed signs of incipient failure. At its north end it was recommended that the trench could be drained under the new highway into one of several natural drainage courses between Stas. 54+00 and 56+00.

Based on detailed mapping of the cut, and an indicated depth to bedrock of 86 ft (26 m) below grade as determined from seismic data, the project soils engineer undertook an investigative program in this cut to determine a stabilization scheme.

Fault Zones - Three faults or fault zones (Bergendahl and Koschmann, 1971) cross the projected alignment. Unstable ground conditions existed in all three of these areas but no direct correlations could be demonstrated between this instability and the faults. The absence of any definitive surficial evidence of these faults implies that they are inactive.

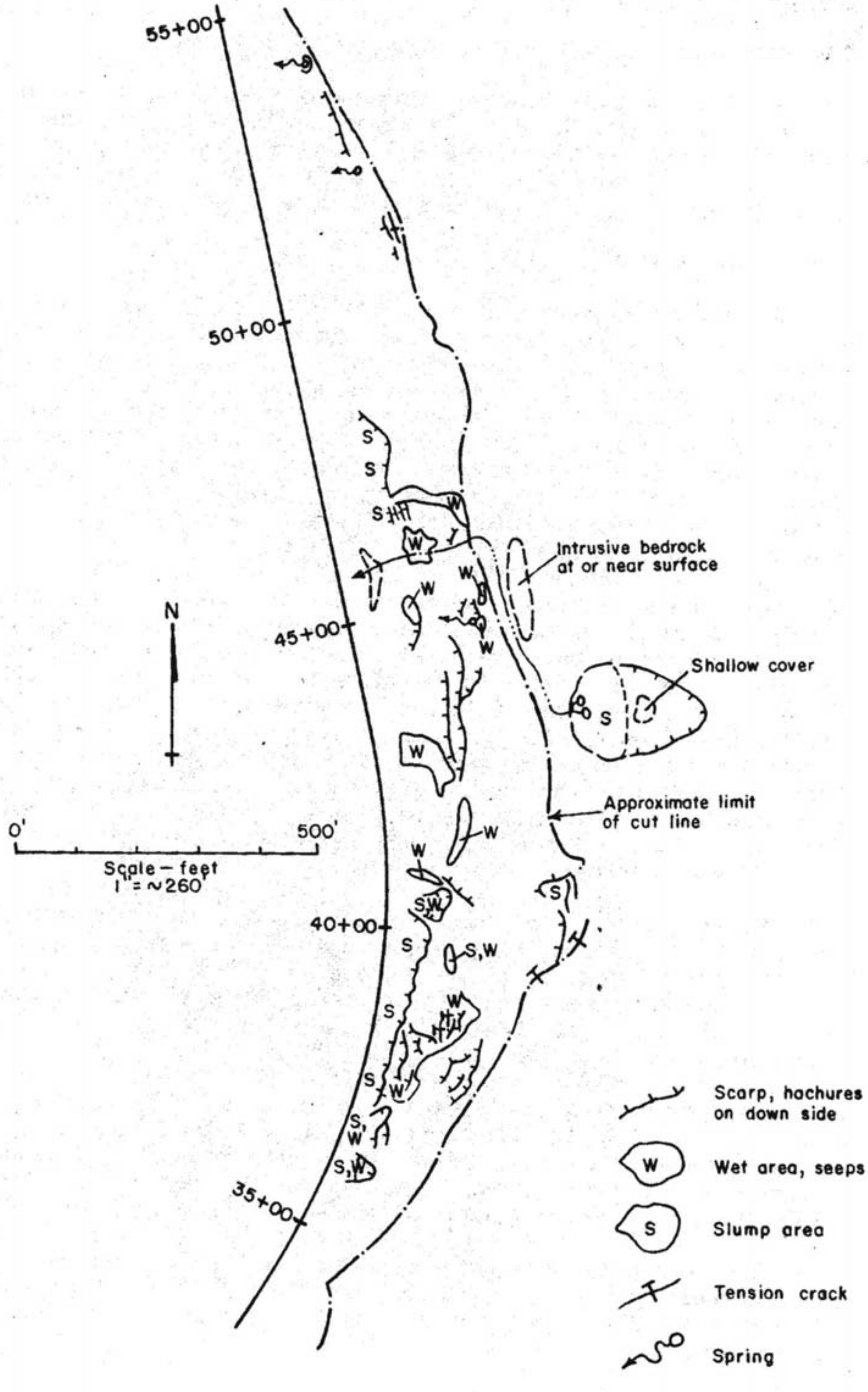

Figure 13. Generalized map of Borrow Area A in early September 1976.

The trace of the unnamed southernmost fault is shown in Figure 2, near station 50+00. Downhill from where it crosses the alignment, the fault was marked by numerous seeps and springs, hummocky ground, and small-scale failures in cuts, fills and natural slopes. This area was apparently part of an old landslide. If this fault was exposed in bedrock cuts, it could be a source of water and an engineered drainage plan may have been needed to correct any adverse water problems.

The Mayflower fault intersects the alignment as shown in Figure 2. A zone 300 ft (91 m) wide on the northeast side of the fault included a large area of spongy, hummocky ground with many seeps, leaning and “pistol-butted” trees, and other signs of surface instability. A series of pronounced arcuate breaks in slope, which appeared to be old landslide features, was also noted in this zone. This fault passes under the floodplain of Mayflower Creek where a thick embankment was planned. It was concluded that any minor motion along this fault, however unlikely, would probably be compensated for in adjustments of the fill material.

A third fault crosses the alignment between Sta. 165+00 and 166+50 (Figure 11). A 12-foot (3.6m) scarp, a large boggy, mossy, unstable area with numerous “pistol-butted” trees and a large area of slope wash were observed within and adjacent to the fault zone. This slope wash is the only major breach in the lateral moraine along Tenmile Creek.

Tenderloin Area - An area of extremely unstable surface conditions had been known for several years (Klauber, 1975) and had been studied along the alignment from approximately Sta. 115+00 to Sta. 157+00 (Figure 11). Figures 14, 15, and 16 illustrate density of vegetation cover and unstable morainal materials in this area, which has been nicknamed the “tenderloin.” Numerous scarps and related evidences of failure, as well as dozens of seeps, springs and small boggy areas filled with lush, water-loving vegetation indicated the general instability of this reach of the alignment. The surficial deposits in this area appeared to be a relatively thin, highly irregular mantle of till and colluvium.

The ground surface in the “tenderloin” was highly disturbed, most likely the result of mass-wasting processes. These processes caused the saturated unconsolidated material lying on oversteepened slopes to move downhill. In an effort to determine the nature and magnitude of this downslope movement, the Colorado Department of Highways installed inclinometer test holes at Stas. 124+30 and 138+50 in December 1975. Downhill deflection in these holes was monitored bimonthly from the last half of May 1976 through the first half of September 1976 (Colorado Division of Highways, 1976a). The data generated from this program was erratic, and it was decided that no further evaluation should be made unless more reliable information was collected during the field mapping.

Slope failures in this area had occurred on surfaces with slopes as shallow as 4:1 and some scarps were measured with as much as 35 ft (11 m) of relief. More than 50 small-scale failures were identified from Sta. 115+00 to 160+00 within the mapped zone. Much of this area had natural slopes of 2:1 that had not failed. This stability was in part due to the cohesive nature of the vegetative matte.

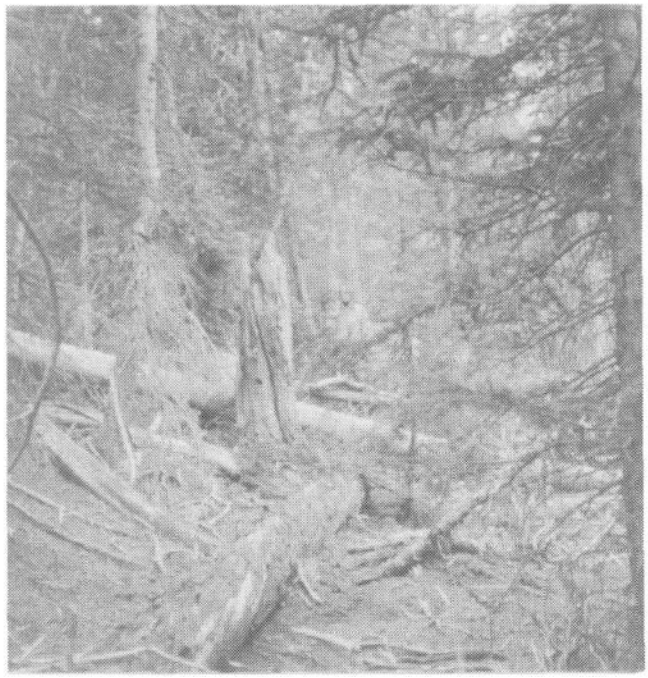

Figure 14. Dense vegetation in the “tenderloin” area.

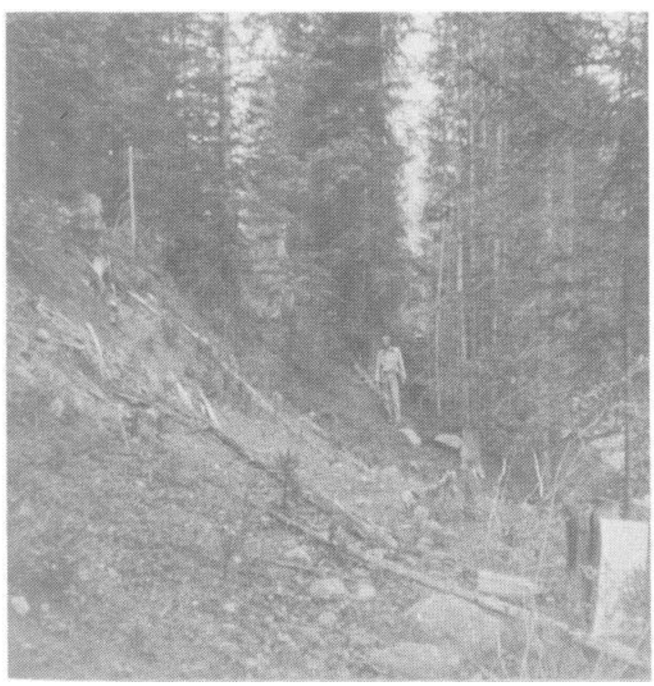

Figure 15. Unstable slope in morainal deposits, “temderloin” area.

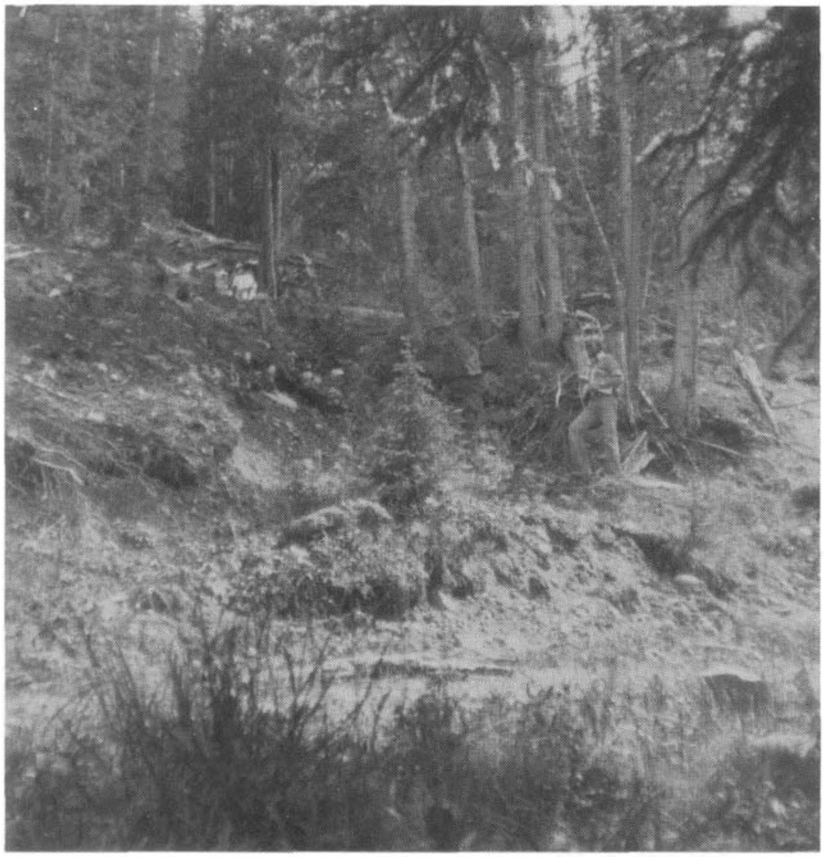

Figure 16. Multiple scarps above an old drill location access road at about Sta. 145+00 in the "tenderloin" area.

Depth to bedrock was of greater concern in the "tenderloin" than in any part of the alignment. In order to make an estimate of this depth, geophysical and test pit programs were conducted to supplement information acquired from surface geologic mapping and from the few exploration test holes. Figure 17 shows the results of this effort.

The possibility was considered that some of the excess moisture in the "tenderloin" was derived from No 1 Canal 1000-1500 ft (305-457 m) uphill. It was confirmed from the owner, however, that no water was released through this canal in 1976.

Talus Train - A talus train was mapped across the alignment from Sta. 180+00 to 182+50 (Figure 11). This deposit included large angular to subangular cobbles and boulders in a highly unstable condition. Many of these boulders were large enough to present a hazard during construction.

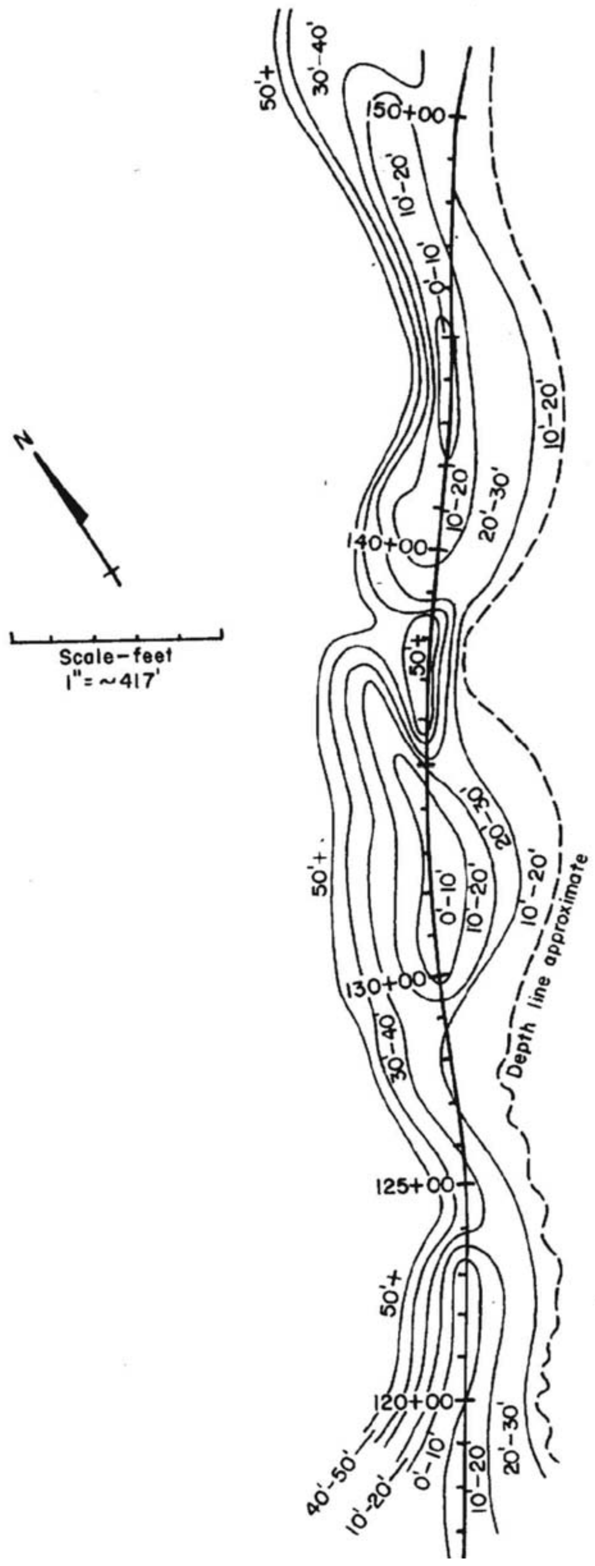

Figure 17. Depth to Bedrock Map, Tenderloin Area.

Oversteepened Morainal Slopes - Lateral moraines that occur along Tenmile Creek in the reach from Stas. 198+00 to 206+00 were in a naturally oversteepened condition with slopes up to 60° (nearly ½: 1) measured during the field mapping. These slopes on thick unconsolidated deposits were identified as a possible construction hazard particularly if undercut when wet. Excavation of these steep morainal slopes was recommended in the late summer when moisture conditions should be most favorable to the stability of the material.

Mine Subsidence - Several old abandoned mine workings were on the new alignment around Sta. 96+00 and Sta. 135+00 (Figure 11). These areas were potential sites for ground subsidence and consequent failure of, or damage to, the road surface. It was concluded that the deep cover gained with short distances above the old mine workings should minimize the subsidence potential.

CONCLUSIONS AND RECOMMENDATIONS

The following types of problems or potential problems were found along the alignment:

1) Landslide and slump on both disturbed and undisturbed ground and in cut-and-fill.
2) Shallow perched water tables causing soft ground with attendant instability problems expected during and after construction.
3) Excessive surface runoff with associated flooding and erosion.
4) Subsidence of the roadway due to undermining.

The following problem areas were identified and addressed:

1) Borrow Area B (Sta. 35+00 to Sta. 54+00). A high cut slope in glacial till has undergone and was undergoing failure due to excess moisture in clay-rich unconsolidated material. It was recommended that engineered drainage be required in this area.
2) Fault Zones – Three faults or fault zones that have been mapped cross the projected alignment. The absence of surface evidence of faulting implied that these faults were inactive and no problems were likely to be associated with them. Structurally unsound materials may be associated with the faults in local areas and this material will require stabilization or removal and replacement.
3) "Tenderloin Area" (Sta. 115+00 to 157+00) – This was an area of extremely unstable surface conditions related to excess moisture in unconsolidated material. Over 50 small-scale failure features were mapped some of which occurred on slopes as shallow as 4:1 and with as much as 35 ft (11 m) of relief. The vegetative cover in this reach provided a substantial cohesive element as well as protection from erosion by surface runoff.
4) Talus Train (Sta.180+00 to 182+50) – This reach included a talus train comprised of many large cobbles and boulders.
5) Oversteepened Morainal Slopes (Sta. 198+ to 206+00) – Oversteepened slopes up to 60 degrees (nearly ½:1) were found. It was recommended that these deposits should

be excavated in small lifts with relatively small equipment during favorable ground moisture conditions.

6) Mine Subsidence – Old abandoned mine workings lie close to the projected alignment in the vicinity of Stas. 95+00 and 135+00. It was recommended that any potential for ground subsidence in these areas should be corrected before emplacement of embankment.

Surface mapping and subsurface data indicated that the depth to bedrock varied from the ground surface to greater than 100 ft (30 m) in some locations. Test pit and seismic information were reasonably complementary and demonstrated a simple, relatively uniform bedrock surface.

Drainage was concluded to be critical in the stability of most of the surficial material on which the highway would be founded. Detailed, site-specific drainage design was recommended in critical locations.

Till and colluvium cover large parts of the alignment. These deposits should be stable in well-drained areas but relatively unstable in the presence of excess moisture. Locally these deposits have failed by slumping or sliding on slopes as shallow as 4:1. Much of this material stood in an oversteepened condition (2:1) by an overlying stabilizing vegetative matte. Disturbance of this matte reduces slope stability.

It was anticipated that both competent and incompetent bedrock probably would be encountered in cuts along the alignment. The orientation and spacing of bedding, joint, fracture and foliation planes and the presence or absence of water determined the degree of safety or hazard in any individual cut.

The absence of lenses of clay or organic matter in the test hole drilled in Mayflower Creek indicated that the thick alluvial deposit would support the extensive artificial fill after the surface was cleared.

Much of the excavated material along sections of cut was reckoned to be acceptable as embankment fill. Side-casting of unstable zones of clayey material, organic matter, and fault gouge was recommended.

Due to the inconsistent and erratic nature of the inclinometer hole data collected by the State Highway Department, it was concluded that this data could not be readily evaluated.

The nature of the geology along this alignment was such that many field decisions would have to be made during construction. This was expected to be particularly true of the “tenderloin” area.

ACKNOWLEDGMENTS

This project was undertaken by the authors and by Jeffrey L. Hynes, then employed at Amuedo and Ivey. Each of these persons participated in the field and office aspects of the project. During the project, considerable help was provided by Messrs. Warren Alloway, William Klauber and

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EPILOGUE

Construction on the realignment began in the spring of 1977 under the supervision of the Colorado Department of Highways. The highway was placed in service in September 1978. The limited construction season forced a shutdown from late 1977 until late spring, 1978.

In early 1979, Centennial Engineering, Inc. submitted the design of this highway to the Colorado Consulting Engineers Council in a competition for the best design and construction project, completed in 1978, for under $5,000,000. Centennial won this competition and received an award for it in May 1979.

During the construction period, monthly inspections were made by the design engineer, the soils engineer, and the engineering geologist. We had concluded in our project report that many field decisions would have to be made, and this prediction proved to be true, particularly in the "tenderloin" area.

The two problem areas of most concern in the initial investigations had been the Borrow Area B cut slopes, and the "tenderloin" area. Inspections concentrated on Borrow Area B, and later on the "tenderloin." On July 31, 1982, an informal inspection trip was made and a series of pictures was taken along the alignment.

Figure 18 shows Borrow Area B and the stabilizing berm, which was emplaced to load the base of the cut. The crest drain (Figure 19) along the top of the cut was performing as intended although some material has sloughed down into the permeable drain. Some slumping (Figure 20) had occurred in the vicinity of Sta. 45+00 and this appeared to be due to excess water in a clayey environment.

Climax planted several thousand seedlings in various cuts along the alignment in addition to hydromulching all cuts. Figure 21 shows the hydromulching under way during November 1977, with snow on the ground. Although the evergreen seedlings grow slowly, the hydromulching has been effective as witnessed throughout all of the alignment where bare rock is not exposed.

The system of subdrains placed in the "tenderloin" area was of particular interest. Figure 22 is a panoramic view of the "tenderloin" in which it can be seen that no major failures have occurred. Minor slump features such as the one shown in Figure 23 are uncommon through this reach of the alignment.

In summary, it is apparent that about four years after the highway was put in service no failures of consequence were apparent. Therefore, this alignment is a good example of what intensive engineering geologic analysis, soils engineering, and design engineering can accomplish with responsive construction supervision.

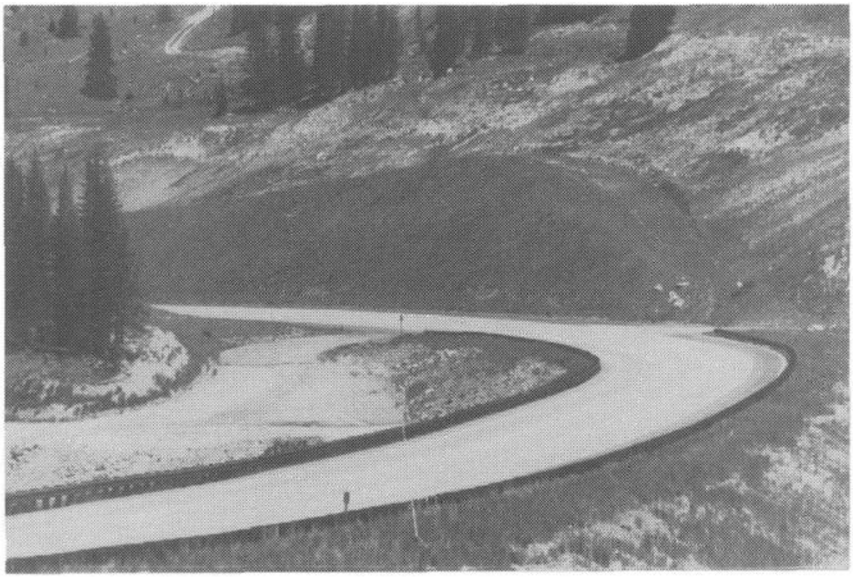

Figure 18. Borrow Area A. Berm (dark arcuate area) in lower part of cut has been an effective barrier to major failure.

Figure 19. Crest drain on right above Borrow Area B cut. Drain is effectively transmitting water through porous blanket although some sloughing has occurred.

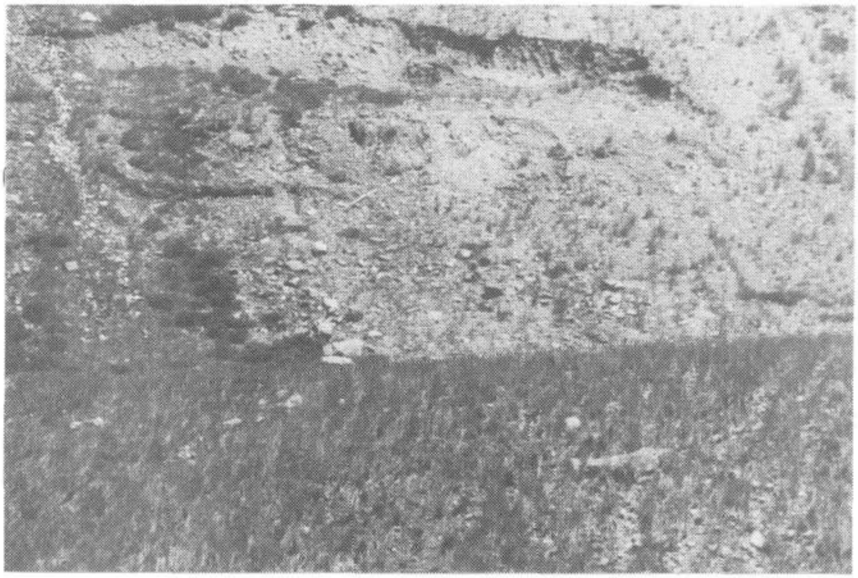

Figure 20. Minor sloughing of morainal material in Borrow Area B above berm surface (lower third of picture).

Figure 21. Hydromulching, November 1977.

Figure 22. Panoramic view of “tenderloin” area. Note the well-established stand of grass in the foreground. Rocks such as the large boulder to the left were place at random along some of the slopes as a matter of aesthetics.

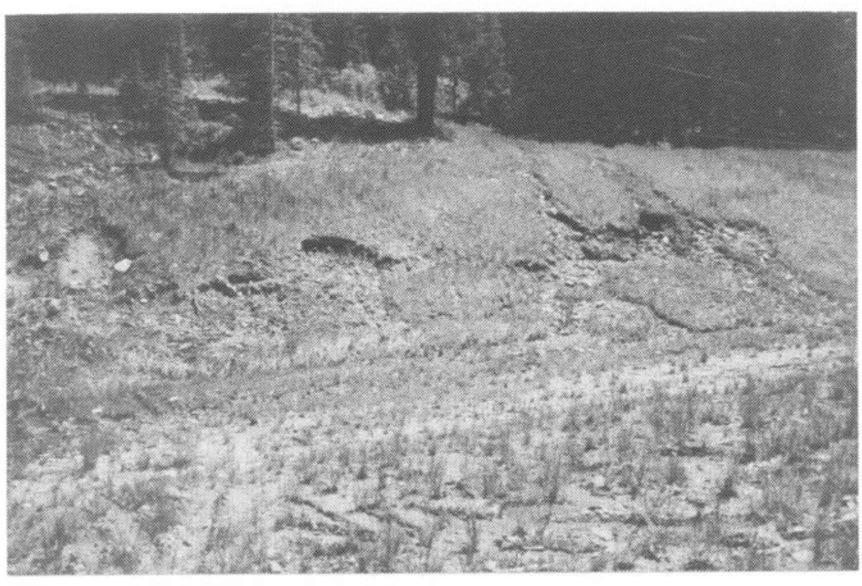

Figure 23. Minor slump features in the “tenderloin” area just south of Humbug Creek.

US HIGHWAY 40, EAST BERTHOUD PASS

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Key Terms: moraine, till, talus, ground nails, landslide, groundwater, alpine glacier

ABSTRACT

US Highway 40 road improvements near the summit of Berthoud Pass in Clear Creek County, Colorado, included adding an uphill passing lane, paved shoulders, and snow storage and traffic safety barriers. The work involved stabilizing an active landslide, rock excavation and installing cut and fill retaining walls in rock and alpine glacial till. The paper provides a description of the geology located within the project site and an engineering evaluation of the ground materials.

INTRODUCTION

In the spring of 1999, Yenter Companies, Inc. was awarded a $37.4 million dollar contract by the Colorado Department of Transportation (CDOT) to improve nearly 3 mi of US Highway 40 east of the Berthoud Pass summit and continental divide. US Highway 40 is a primary traffic corridor connecting north central Colorado to Interstate Highway 70 and is located approximately 65 mi west from Denver (Figure 1).

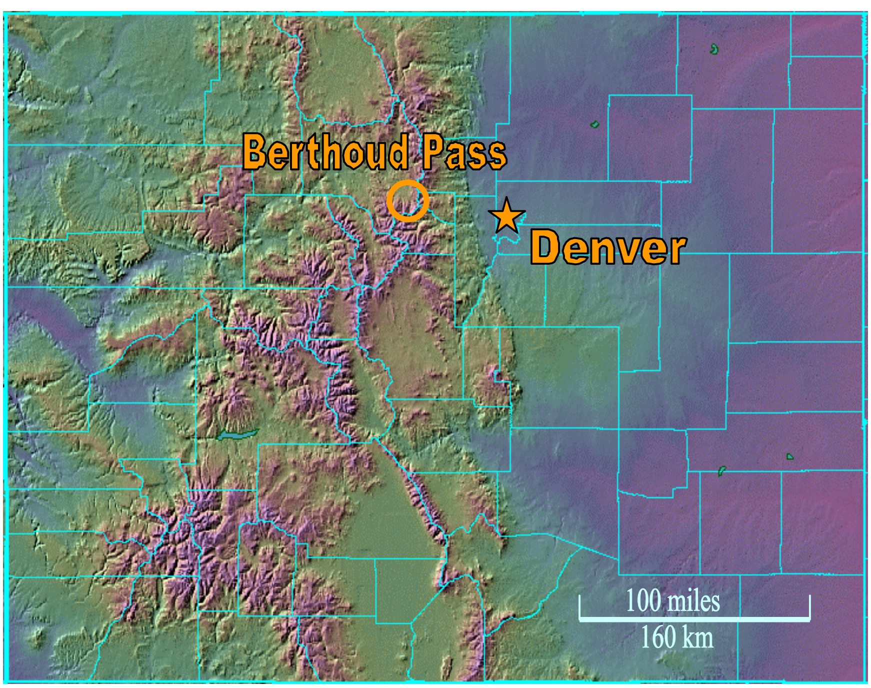

Figure 1. Shaded relief map of Colorado showing an outline of the counties. The proximity of Berthoud Pass relative to Denver is indicated.

For numerous mountain towns such as Winter Park, Granby, Grand Lake, Fraser and Tabernash, US Highway 40 is an important economic link providing the means for tourism, goods and essential services. Recent traffic count records indicate that the annual average daily traffic count is 4,476 cars and trucks (CDOT, 2002). However, this count is expected to increase sharply in the future because of the newly completed road improvements.

In an effort to expedite project delivery, the CDOT accelerated the preliminary design phase and included design/build elements into specific portions of the contract. As such, some design elements, including the geologic/geotechnical investigations were not fully completed until the construction phase. The pressures of successfully completing a difficult construction in the mountains both timely and efficiently, while conducting site explorations that have a direct bearing on the scope of construction activities, forced a greater degree of adaptability on the part of the engineering geologist and design professionals than that required on other, more familiar design-bid-build projects.

HISTORY OF THE CORRIDOR (HARRISON, 1974; HUNTER, 1998)

The development of a modern passage over the continental divide in what is now referred to as Berthoud Pass probably began in the 1840's by trappers who followed Indian paths and game trails, widening the crossing, as needed, to accommodate pack burrows. Around the same time a permanent trading post was established on the west side of Berthoud Pass near the town of Fraser. Improving the pack trail to a wagon road likely occurred some years later as traffic and interest in the area increased during the 1860 gold rush in Clear Creek County. The pass's namesake, Captain E. L. Berthoud, was a civil engineer from Golden, Colorado. He is credited with discovering the pass that reaches 11,307 ft (3,426 m) at the summit, on May 10, 1861, while seeking a western mail route. The state bought the road from the Middle Park Wagon Road Company in 1891 and proceeded to provide additional improvements and maintenance to the wagon road until 1911 when the publication, The Republic wrote on December 31, 1911 that, "The highway (over Berthoud Pass) is now passable even for low-powered autos." (Figure 2).

In 1920, the Bureau of Public Roads, the Highway Department of Colorado and the US Forest Service undertook a major construction/realignment project. A budget of $220,000 was appropriated to upgrade 7.6 mi of roadway on either side of the pass. Major road improvements consisting of widening the roadway to 16 ft, flattening grades to a maximum of 6% and installing a state-of-the-art drainage system required a new alignment. Concurrent work began on either side of the pass and continued for 2 seasons. At the completion of the second season approximately 16 mi of roadway were improved with a minimum road width of 18 ft for a total cost of $506,000. Intermittent maintenance and improvement projects continued throughout the years with the road alignment remaining essentially unchanged.

The latest public works project awarded by the Colorado Department of Transportation in the Spring of 1999 and completed in the Fall of 2002 succeeded in accomplishing multiple safety, recreational and environmental goals while further widening the road from 22-ft at the narrowest to a minimum of 66-ft.

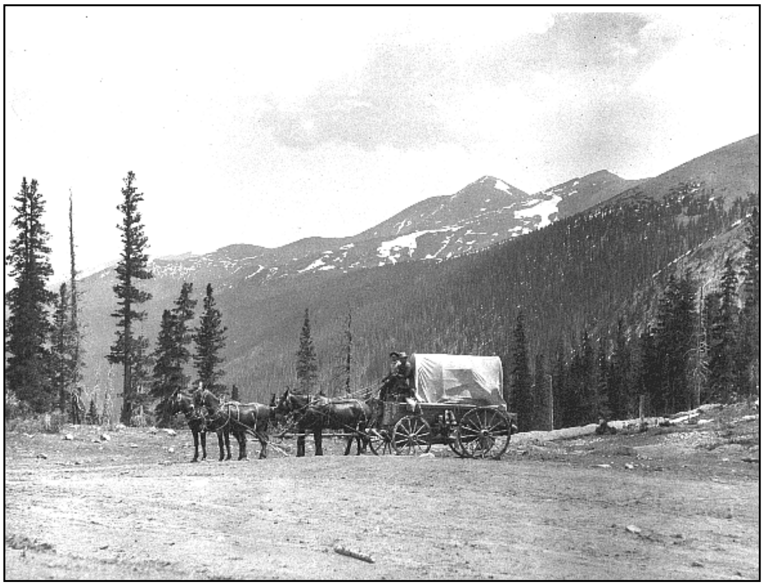

Figure 2. Survey crew in covered wagon at summit of Berthoud Pass. [Campbell, ca 1905]

GEOLOGY

Figure 3 is a 1933 oblique air photo of Berthoud Pass looking north. Today the road alignment follows essentially the same route as seen in this photo. Portions of the US Highway 40 roadway that were widened during the recent project are partially shown in the foreground and include all

Figure 3. Aerial oblique of Berthoud Pass and the continental divide looking north. Summit can be seen just below the center right of the photo. In the midst of such steep and rugged terrain, the two prominent glacier cirques in the foreground provided a tempting opportunity to switch back the road (T.S. Lovering, USGS Geologist ca 1933).

of the sharp switchbacks shown in the photo. As this early photo clearly shows, the terrain is dominated by glacial landforms. The slopes adjacent to the roadway steepen to nearly 40 degrees at the lower elevations of the project. Notice that the road switches back in the glacial cirques where the slopes are flatter. The two glacial cirques in the foreground mark the origins of Pinedale Age Glaciers that were active in the project area over 12,000 years B.P. The two prominent glaciated valleys below the Berthoud Pass Summit are seen trending parallel to one another and are oriented in a southeasterly direction. The soils encountered in the project site consist primarily of immature glacial drift mostly ice-laid with minor amounts of sediment deposited by melt water. The remnants of what was likely a hanging valley in the darkened shadow of the photo, referred to as Floral Park, provided a highly prized and protected alpine wetland habitat. Along the highway in Floral Park the steep wall of the glacial valley with a northwestern exposure is a protalus rampart. These thick (approximately 10 m) deposits of scree formed nearly parallel ridges that are almost fully concealed by a dense, evergreen forest supported by a thin layer of topsoil. Figure 4 is a photo of the 1905 wagon road that traversed the area and shows unstratified well-graded sediments ranging from boulders to fine silts and clays.

Figure 4. Wagon Road towards Berthoud Pass. Note the unstratified glacial drift in road cut (Campbell, circa 1905).

A simplified geologic map of the area is shown in Figure 5 complete with faults and rock outcrops mapped by Theobald (1965). The 4 switchbacks are labeled 1 through 4 from the lowest elevation to the highest. At switchback 1, Hoop Creek is located in a deeply incised stream channel trending along the prominent fault line (Figure 6). Although the switchback appears to be located in a glacial trough it is actually located on a steep till slope. The 1920 road builders built a switchback that left a steep (37 to 45 degrees) cut slope, 82 ft (25 m) in height. This area became a major rockfall hazard exposing the roadway to significant rockfall risks during periods of inclement weather. To mitigate the rockfall hazard, retain the eroding slope and provide additional roadway turning radii in the tight switchback, a three-tiered (triple tier) ground nail wall was constructed onto the slope. Ground nail walls are a type of earth retention system that is constructed in staged excavations. At each excavation stage (usually 1 to 2 m)

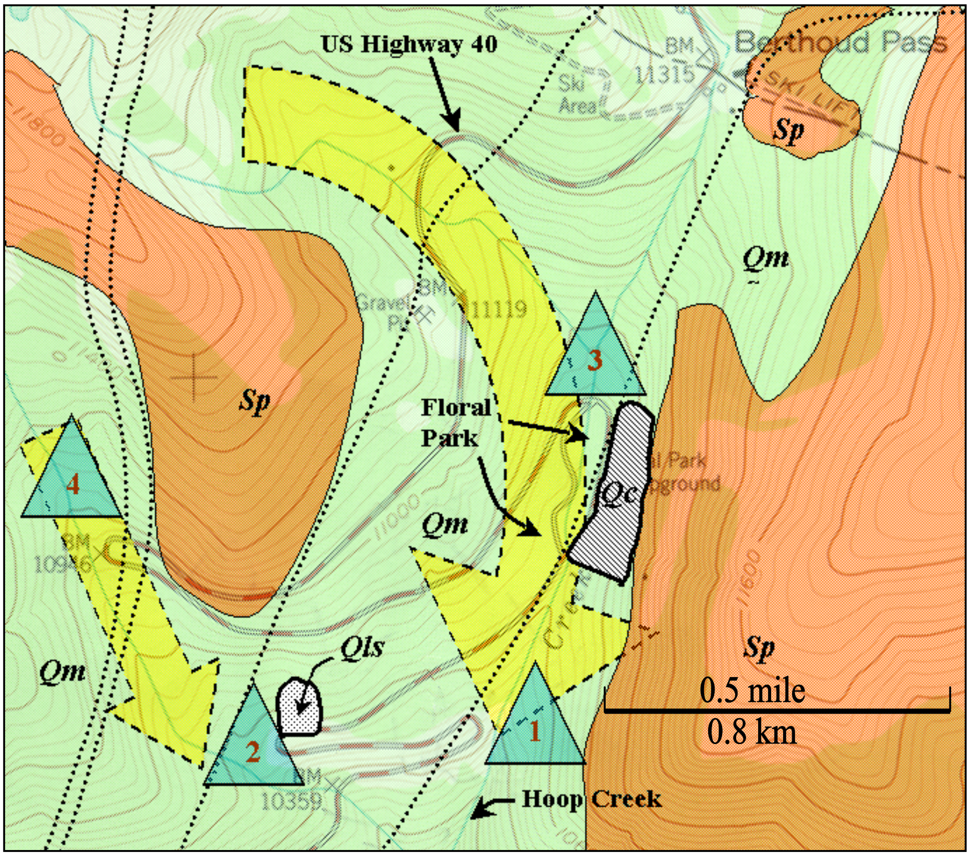

Figure 5. A simplified geologic map of the project area. The large open arrows indicate paths of Pinedale Age Glaciers. “Sp” are outcrops of Silver Plume granite, gneiss and/or migmatite. “Qls” is a recent landslide. “Qc” is a quaternary colluvium protalus rampart. “Qm” is comprised of unstratified glacial drift containing some meltwater deposits. The dotted lines represent concealed faults mapped by Theobald (1965). The switchbacks are numbered with triangles one through four as they are encountered while traveling up to the summit.

steel bars are installed in grouted drill holes (ground nails) spaced at a designed frequency extending back into the slope. Reinforced shotcrete (pneumatically placed concrete) connects the series of ground nails at the face of the excavation. Once the ground nail wall was complete, a hybrid mechanically stabilized earth (MSE) wall was built in front of the ground nail wall to comply with the aesthetic requirements of the project. With a maximum overall height of 85 ft, the triple tiered ground nail wall at Berthoud Pass is the tallest ground nail wall in Colorado and is one of the tallest in the country.

Switchback 2 is within a glacial trough. Adjacent to the switchback is a landslide that slipped 3 to 6 ft (1 to 2 m) during the July 4, 1996 weekend (Figure 7). Evidence from two inclinometers indicated that the slip surface occurred at the overburden/bedrock contact. Seven rows of permanent tiebacks (146 anchors in all) were installed within the slope and buried. Each tieback consisted of a 9 strand high strength steel wire tendon installed up to 90 ft into the slope in a grouted hole through a precast reinforced concrete panel 10 ft wide by 10 ft tall by 18 in thick (3 m x 3 m x 0.45 m). Once the grout encapsulating the tendon cured, each tendon was post-tensioned to 135 tons (1,200 kN). In addition to the tiebacks the landslide also benefited from a

Figure 6. Aerial photo showing Berthoud Pass switchbacks. Fault lineation aligned with Hoop Creek is prominent as are numerous snow avalanche paths on either side of glacial valley. Poorly developed hanging valley is located near switchback 1 (see Figure 5). [USDA, ca 1991].

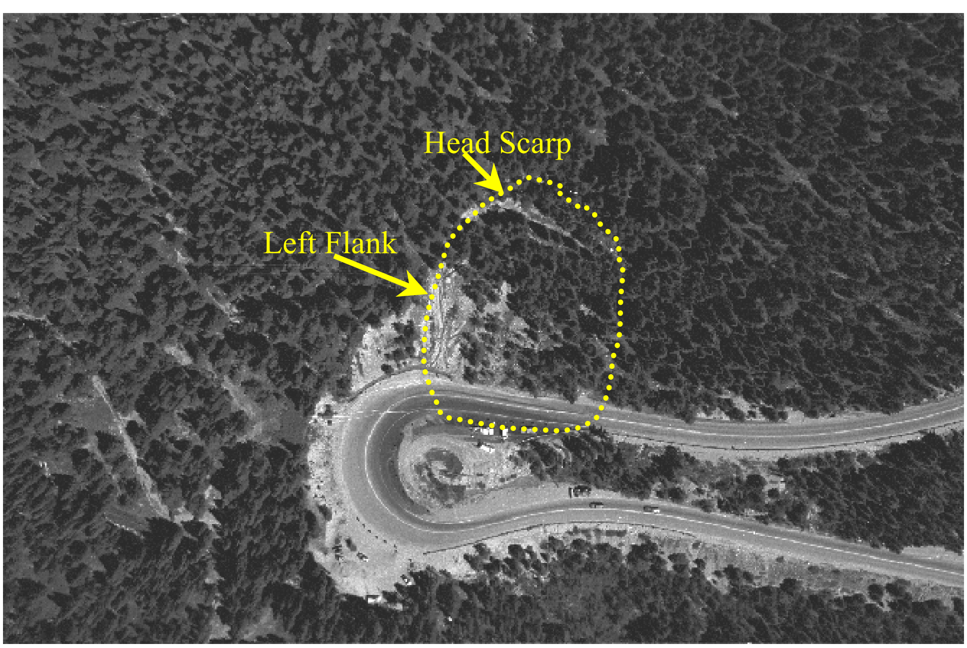

Figure 7. Landslide activated on July 4 weekend 1996. Pavement sheared and heaved as slide moved up to 6 ft (2 m) [CDOT, ca 1996].

two tiered hybrid mechanically stabilized earth (MSE) wall installed near the roadway to flatten and buttress the regraded slope (Figure 8). Prior to and throughout construction a steady flow of water sprung from the slope midway up the slide and ran along it's left flank (Figure 7). The flow occurs at the trace of the slip plane on the ground surface. Groundwater encountering the slip plane likely follows the zone of disturbed dilated soil until reaching the ground surface. The author observed this same stream of water flowing down the slope prior to 1996 while traveling over the pass. It appeared as a spring on the hillside. From a field inspection of the slope shortly after the 1996 slide event, it became apparent that this "spring" flowed along the left flank of the landslide. The right flank traces through the heavy forest and did not contain a stream of water.

Figure 8. Completed roadway showing landslide area. Seven rows of permanent tiebacks are buried in the slope. The tiered hybrid MSE wall transitions smoothly into another hybrid MSE wall façade [CDOT, ca 2002].

Near switchback 3 a significant deposit of scree complicated construction in two ways. Groundwater flowing from the scree would surge soon after rainfall events flooding the traffic detour and adjacent work areas and grout containment for ground nails within the scree was difficult. Extensive underdrains were installed adjacent to the scree deposits to intercept the surging groundwater and channel it into the Floral Park wetland habitat (Figure 9). Hoop Creek crosses the highway and enters Floral Park at switchback 3.

The roadway immediately above and below switchback 4 encountered highly fractured and sheared bedrock. The rock cut above the switchback was laid back at a safe cut slope angle and draped with rock fall mesh as an additional safety precaution. The road cuts below the

Figure 9. View of switchback 3 looking into Floral Park. Wetland habitat is located inside the hairpin turn. The scree is deposited against the mountain slope on the viewer's left. A small portion of the scree is visible high up on the mountain [CDOT, ca 2002].

switchback were stabilized with ground nail walls. Again containing grout within the dilated and highly sheared rock discontinuities was difficult. The switchback itself is located in a glacial trough, which because of the flatter grades reduces the amount of road cut and fill; however, these locations increase the likelihood of encountering significant groundwater. Note that switchback 2 contained a groundwater-induced landslide and switchback 4 encountered large volumes of water flowing from the excavation.

HYDROLOGY

The main drainage channel in the area is the Hoop Creek drainage, which trends along a prominent fault line and glacier trough. Smaller feeder creeks trending along the upper glacial troughs merge with Hoop Creek below the project. Subsurface water encountered during construction throughout the project was extraordinary. Most of the groundwater appeared to follow buried stream channel deposits trending along the glaciated valley troughs. However, seeps and occasional springs were encountered throughout the project and appear to be controlled by fracture flow conditions in the bedrock. In the Floral Park area a surge of groundwater would lag storm events by a few days as the subsurface flow would recharge, make its way into the scree deposits and eventually empty onto the highway. Piezometers slightly above the landslide area indicated that the elevation head adjacent to the slip surface was approximately 5 ft (1.5 m) above the crest of the landslide. During installation of tieback anchors to stabilize the landslide a borehole produced a continuous flow of water of approximately 30 gpm (135 liters per min) with a 17 ft (5 m) drop in the water level measured at the piezometers.

CHARACTERIZATION OF ENGINEERING SOIL PROPERTIES

Approximately 294,000 sq ft (27,000 sq m) of cut and fill retaining walls were constructed to allow the roadway widening (CDOT, 1999). The 36 retaining walls, some stacked above a base wall in a tiered configuration, would stretch end-to-end for approximately 18,100 ft (5,500 m) (CDOT, 1999). During the original site investigation of the project a total of 41 test borings and 22 seismic survey lines were completed (Pardi, 1998). The seismic lines were laid out perpendicular to the roadway for short distances in an attempt to determine the depth to bedrock. However, the data often conflicted with the borehole information, probably due to early seismic reflections on sloping bedrock surfaces. Therefore the borehole frequency averaged 1 hole per 440 lineal ft (135 lineal m) of wall layout. In the landslide area where CDOT had conducted a subsurface investigation 2 boreholes were used to determine the subsurface stratigraphy and bedrock surface even though the proposed 4 tiered retaining walls relied upon anchoring permanent tieback tendons into bedrock (Andrew, 1998).

Gradations taken from split spoon samples indicated that the glacial till is well graded with approximately 15% fine content. The fines were predominantly non-plastic silts with a few samples containing lean clay. Oversized particles of cobbles and boulders probably represented 15 to 20 percent of the till by weight. The fine sand sized particles were observed under low magnification to possess a subangular shape. The blow counts from the standard penetration tests were variable and ranged from 4 to refusal. On average, however, the blow counts indicated that the till was medium dense to dense. CDOT collected disturbed soil specimens from each wall and conducted direct shear tests on material finer than 4.75 mm. The post-peak shear strengths on samples prepared at 90% of modified proctor averaged 38.1 degrees for the angle of internal friction and a 1.6 psi (11 kPa) cohesive intercept. During construction, 3 consolidated-drained triaxial tests were completed on 6-inch (150 mm) diameter specimens compacted to 95% of standard proctor density. The post peak shear strengths (at 15% axial strain) were slightly higher than the direct shear results.

Assigning soil properties for a particular engineering design became a process of reviewing all of the available borehole and laboratory data, examining the specific site for signs of past slope weakness and observing the type of vegetation in the area that may indicate shallow water conditions. If concerns over the soil properties continued during the design team review, further negotiations with the review team commenced and/or a drilling rig would be mobilized to collect additional data for consideration. Generally, consensus with the design review team was achieved without much deliberation. However, at one location where drilling was inaccessible, a contingency plan was included in the reviewed design prior to initiating construction. Once the area was opened to inspection by the review team a consensus decision was reached based on a prearranged decision tree selecting the original design or the contingency.

SUMMARY AND CONCLUSIONS

The US Highway 40 widening project over Berthoud Pass was challenging. To complete the project at the pace demanded by the CDOT, adjustments to a traditional project delivery program had to be made. Towards this end preliminary design work, including supplemental subsurface investigations, design changes, and multiple, alternate designs were completed during construction. In general, the earthen materials were quite strong; however, large seepage forces, particularly in the roadway switchbacks were initially underestimated. The 1920 roadway realignment project constructed three switchbacks in glacial troughs. The recent widening project encountered significant flows of continuous groundwater within these troughs, prompting design adjustments during construction. An on-going evaluation and field verification of the ground conditions and their impact on the design and construction activities evolved as the work progressed. Although working in such an environment produces a sense of immediacy not normally associated with highway design it can be successfully managed as the US Highway 40 project attests (Figure 10).

Figure 10. The completed project with a few labeled landmarks. [CDOT, ca 2002]

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CHEESMAN DAM
TURN OF THE CENTURY WONDER

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Key Terms: Cheesman Dam, Denver water supply, arch dam, masonry blocks

SUMMARY

Cheesman Dam and reservoir have been an integral part of the Denver water supply system since 1905, providing Denver's first mountain water storage and the first substantial and continual storage of water for municipal use in the Rocky Mountain region. Upon completion, Cheesman Dam became the highest gravity-arch stone masonry dam in the world. Cheesman Dam provides an example of early arch dam design and construction as part of the historical development of the city of Denver and its water supply system developed by Denver Water. Cheesman Dam was built by the Denver Union Water Company, the predecessor of Denver Water from 1900-1905. Today, it is owned and operated by the Denver Board of Water Commissioners/Denver Water.

Denver Water provides storage, treatment and distribution of quality drinking water to the Denver metro area. It is a quasi-city agency governed by the Board of Water Commissioners, consisting of 5 members on staggered 6-year terms appointed by the mayor of Denver. The water supply system consists of 18 dams, both concrete and earthfill, 3 water treatment plants, 3 major tunnels (Moffat, Roberts, and Foothills), one reuse plant (in progress) and numerous pump stations and conduits.

Cheesman Dam is located on the South Platte River, 6 miles southwest of Deckers, Colorado and 60 road miles southwest of Denver. It is situated in the southern part of the Colorado Front Range; the dam and reservoir are located entirely in the Precambrian Pikes Peak granite bedrock. The Pikes Peak granite is a medium to coarse-grained igneous intrusive rock that is hard and competent in the dam foundation area. The dam was constructed of Pikes Peak granite blocks that were quarried on site. Even though Cheesman Dam was built after the turn of the last century, when few environmental concerns were expressed, it possesses many features that make it environmentally attractive today. Its construction, featuring native granite, causes it to blend visually with the canyon surroundings, and the spillway design provides for water to cascade naturally down the left rock abutment rather than the dam itself.

The structure is a gravity arch, masonry dam, faced with granite blocks, 18 feet thick at the crest and 176 feet thick at the base (Figures 1, 2, 3, and 4). Today Cheesman Reservoir is the third largest facility in the Denver Water system, after Dillon and Eleven Mile Reservoirs. It has a 79,064 acre-feet water storage capacity and an 18 mile shoreline (Table 1).

Cheesman Dam has contributed significantly to the definition of progress in the development of dam engineering and construction technology in the 20th century. It was designated a National Historic Civil Engineering Landmark by the American Society of Civil Engineers in 1973.

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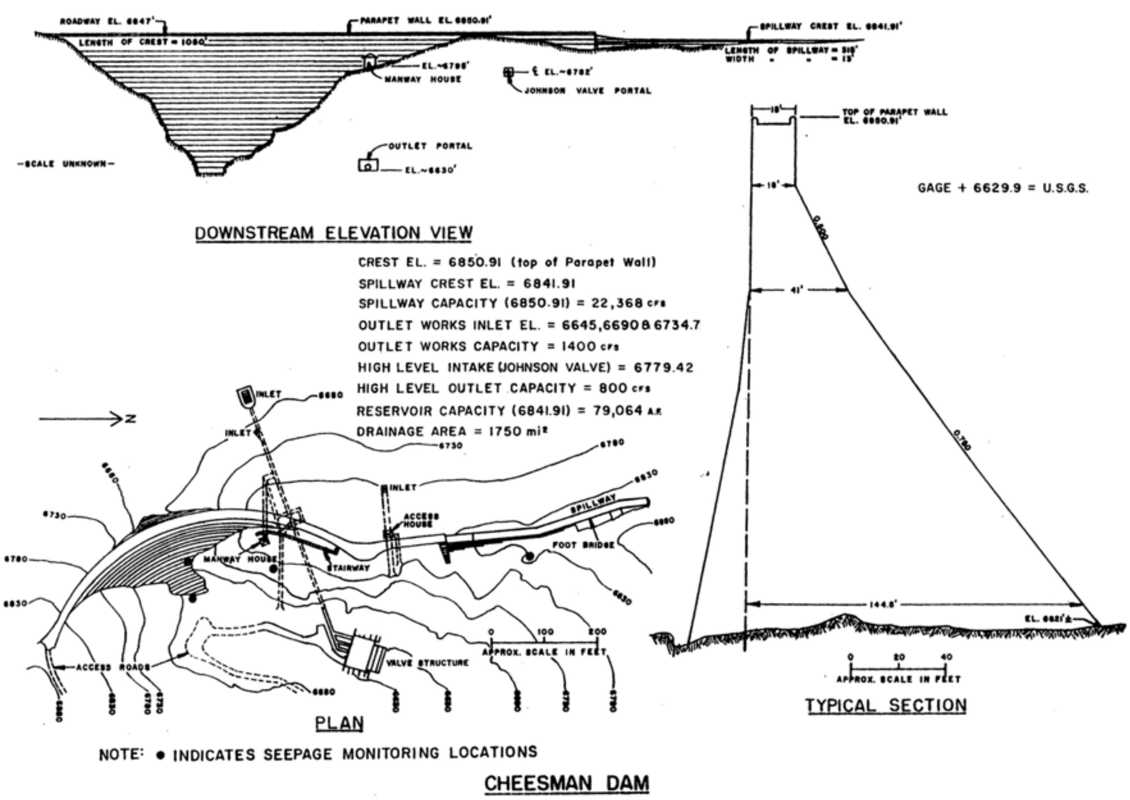

Figure 1. Plan, Profile, and Section of Cheesman Dam.

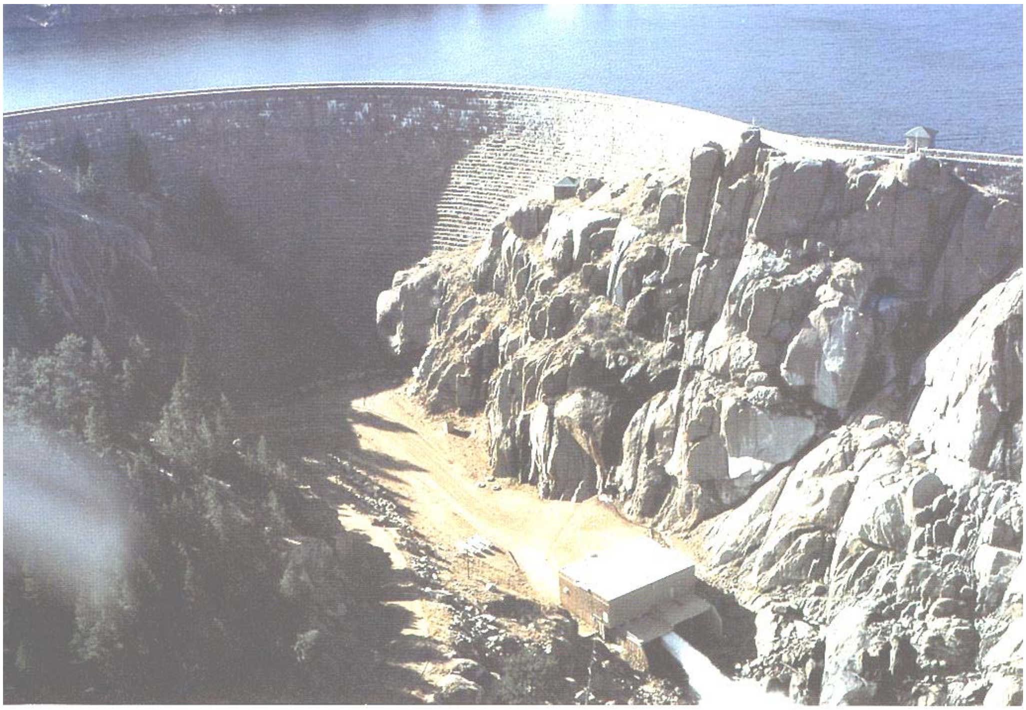

Figure 2. View of Cheesman Dam.

Figure 3. Operation of the Spillway and Upper Outlet Works.

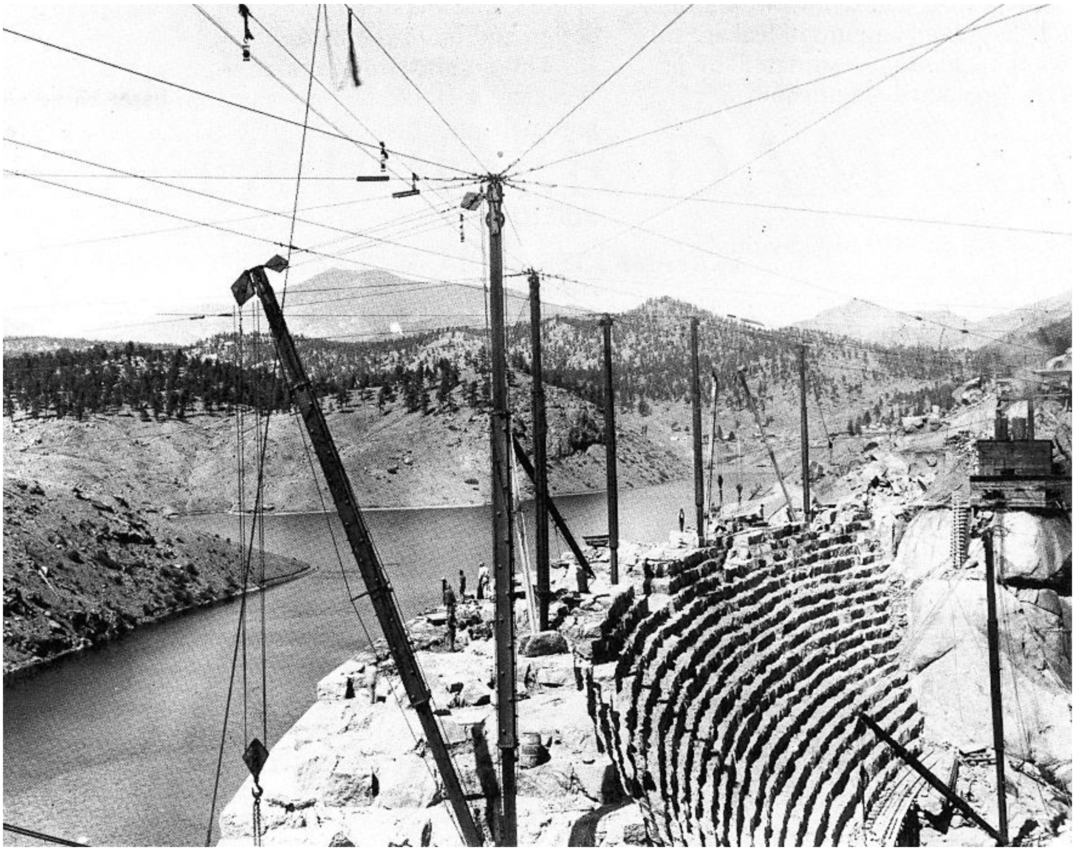
Figure 4. View of Aerial Tramways During Construction of Cheesman Dam.

Table 1. Summary of Pertinent Information for Cheesman Dam

DAM	
Type	Gravity Arch Masonry
Date Constructed	1905
Height	221 feet
Crest Elevation	6850.91[1] (top of parapet wall)
Crest Length	1050 feet (including spillway)
Crest Width	18 feet
SPILLWAY	
Type	Overflow broad-crested weir
Crest Elevation	6841.91
Crest Length	315 feet
Discharge Capacity	22,370 cfs (at elevation 6850.91)
MAIN WATERWAY OUTLET WORKS	
Conduit Length	550 feet
Conduit Size	78” I.D. steel pipe at downstream
Guard Valves	42” gate valves in rock tunnels 3-48”, 2-30”, 1-16”, 1-14”, and 1-10” ball valves
Control Valves	2-42”, 1-24”, 1-12”, and 1-8” free
Inlet Elevations	6645, 6690, and 6734.7
Discharge Capacity	1400 cfs
HIGH LEVEL INTAKE	
Conduit Type	Rock tunnel
Conduit Length	150 feet
Conduit Size	6-foot-diameter
Guard Gate	5’ x 7’ slide gate
Control Valve	62” diameter Johnson needle valve
Inlet Elevations	6779.42
Discharge Capacity	800 cfs
RESERVOIR	
Storage at Spillway Crest	79,064 acre feet
Surface Area at Spillway Crest	874 acres
Drainage Area	1750 square miles

[1]Operations added 4” to 6” of concrete to crest of dam in 1996 and 1997.

THE FAILURE OF CASTLEWOOD DAM

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Key Terms: Castlewood Dam, dam failure, rubble-filled masonry dam, forensic geology, Denver Basin, early dam construction methods

ABSTRACT

Castlewood Dam was constructed in 1890 by the Denver Land and Water Storage Company on Cherry Creek, approximately 30 mi (50 km) southeast of Denver, Colorado. Castlewood Dam was a 70-ft-high (21m) rubble-filled masonry dam and was designed with an optimistic reservoir capacity of 12,000 acre-ft (14,800,000 cubic m). The combined spillways for the dam were estimated to pass 4,000 cubic ft/sec (113 cubic m/s) although Cherry Creek was known at the time to have experienced flows over 10,000 cubic ft/sec (283 cubic m/s). From the time the reservoir was first filled, Castlewood Dam experienced considerable seepage and other structural problems. At one time, an earthfill berm was placed against the upstream face to help control this seepage. In the evening of August 2, 1933, more than 8 in (200 mm) of rain fell in the upper Cherry Creek drainage basin in less than 3 hours, causing the dam to overtop and ultimately fail in the early hours of August 3rd.

The failure of Castlewood Dam released an estimated flood of 126,000 cubic ft/sec (3570 cubic m/s) downstream through the city of Denver resulting in the loss of two lives and approximately 1 million dollars worth of damage. The loss of life would have been even greater; except for the quick thinking and determination of the dam tender, who was at the site when the dam failed and traveled to nearby Castle Rock to warn the residents downstream. The cause of the failure relates to the foundation, the design and the construction methods. Although the designer disagreed vehemently, Castlewood Dam was preordained to fail due to its incompatibility with the foundation, compounded by poor construction practices. Surprisingly, when the dam did finally fail, much less damage was done to the foundation, especially on the right abutment, than could have been expected.

INTRODUCTION

The greatest flood in Denver's history occurred on May 19, 1864 with massive damage and loss of life. Following this major flood event, flood prevention was ever on the minds of the citizens of Denver for the South Platte River and Cherry Creek, the two major waterways running through downtown Denver. Finally in 1889, work began on Castlewood Dam, approximately 30 mi (50 km) southeast of Denver on Cherry Creek, in Douglas County, by the Denver Land and Water Storage Company. The dam was completed in a record time of 11 months and served as a source of irrigation water until its failure on August 3, 1933. In those 42 years, the dam leaked severely and the safety of the dam was continually questioned until excessive leakage required major repairs to the dam between 1898 and 1900. Shortly after that, the Denver Land and Water Storage Company went bankrupt. From then until it finally failed in 1933, the dam was owned and operated by numerous real estate and irrigation companies, usually with the intent of using the dam to sell desirable acreage. When Castlewood Dam failed after several days of rain culminating in a downpour that dumped 8 in (200 mm) of rain in less than 3 hours immediately upstream of the dam, severe loss of life was averted through the quick thinking and determination of the dam tender (Hugh Paine) and the persistence of several private phone company operators. In spite off this excellent early warning, two lives were lost and approximately one million dollars of damage resulted from the ensuing flood (Horan, 1997). Since its failure, the dam has been left undisturbed and is now within the boundaries of the Castlewood Canyon State Park. Today, all that remains of the dam are portions of the outlet works and most of the left abutment section of the dam along with the overflow spillway at the left end of the dam and a very small portion of the right abutment. Little is known about the failure and the direct causes of the failure have not been recorded. To learn from this failure, we must review the design, the construction, the operations and the chronology, together with the foundation geology to obtain a better understanding of the dam failure.

GEOLOGIC SETTING

The geologic setting for Castlewood Dam begins some 66 million years ago (mya) with the second rising of the Rocky Mountains during the Larimide Orogany, spanning the Cretaceous-Tertiary (K/T) boundary (Figure 1). During and following the rise of the Rocky Mountains, large quantities of synorogenic sediments were deposited in the Denver Basin, just to the east of the Front Range. The earliest sequence of strata, termed the D1 deposits, consisted of basin fills of the Denver and Arapahoe Formations and continued until approximately 64 mya, late Paleocene. At this point in the Paleocene, the surface at the eastern edge of the Front Range represented a large, relatively stable pediment. Following this period of rapid deposition in the Denver Basin a distinct red, purple, yellow and orange fossil soil layer or paleosol was variably developed across the ancient landscape. In places, this paleosol is 20-ft (6 m) thick, attesting to a relatively quiet period in the tectonic development of the Front Range and the Denver Basin. This paleosol is associated with the onset of the next series of synorogenic sediments and separates the D1 sequence of deposits from this next younger sequence of rapid deposition, the D2 sequence. The Dawson Arkose, a portion of the D2 Sequence, is interpreted to involve redistribution of the grus mantle derived from erosion of the Pikes Peak Batholith, the Pikes Peak Granite (Raynolds, 2002).

Following the deposition and pediment forming process of the period represented by the Dawson Arkose, a major volcanic eruption in the Sawatch Range approximately 37 mya, Eocene, resulted in the regional distribution along the Front Range of the Wall Mountain Tuff, called the Castle Rock Rhyolite in the area of Castlewood Canyon. This layer averages approximately 20-ft (6 m) thick and occurs on the tops of buttes between Castle Rock and Monument Hill. Subsequent erosion during the periods of intermittent, catastrophic floods, approximately 34 mya, early Oligocene, deposited the Castle Rock Conglomerate in deep canyons cut into the Castle Rock Rhyolite and the underlying Dawson Arkose. In a reversal of topography, these deep canyon deposits now stand out as the high points in the area of southern Douglas and northern El Paso Counties (Morse, 1985). During the subsequent 34 million years, continued erosion has removed the overlying deposits at Castlewood Canyon, leaving the Castle Rock Conglomerate as the cap

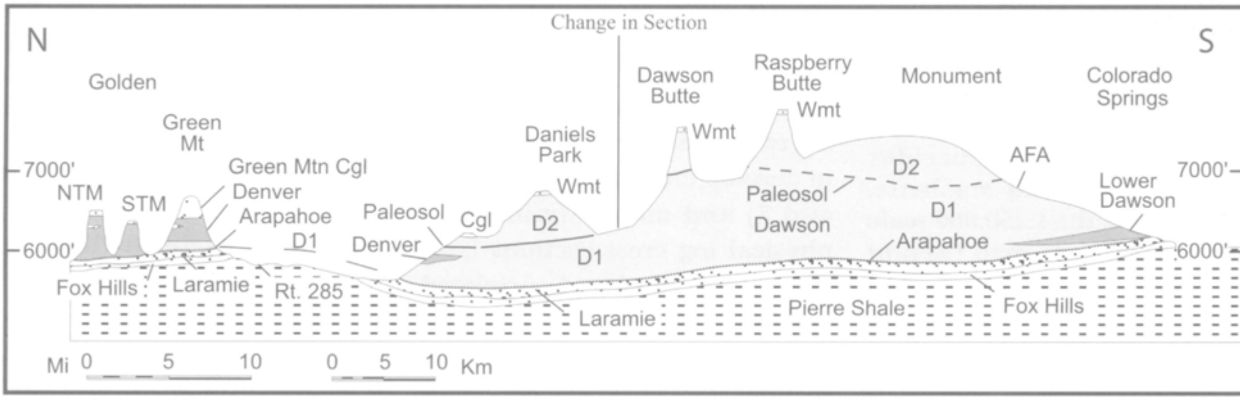

Figure 1. Schematic cross section along western flank of Denver Basin. NTM = North Table Mountain; STM = South Table Mountain; AFA = Air Force Academy; WMT = Wall Mountain Tuff (also known as Castle Rock Rhyolite). Cross section steps closer to mountain front south of where change in section is indicated (Raynolds, 2002).

rock. Erosion, controlled by the regional joint system, has resulted in deep incision through the conglomerate into the Dawson Arkose, forming Castlewood Canyon. Through this process, large conglomerate blocks falling from the higher slopes blanket and partially armor the canyon walls, retarding subsequent erosion. The area immediately upstream of the canyon was sculpted into a broad valley during the period that Cherry Creek was blocked prior to down cutting through the Castle Rock Conglomerate and forming the Castlewood Canyon.

SITE GEOLOGY

Geomorphology

The easterly flow direction of the streams and rivers exiting the Front Range was disrupted when the Castle Rock Rhyolite and subsequent Castle Rock Conglomerate were deposited on the east sloping pediment, forming the Palmer divide in southern Douglas and northern El Paso Counties. Since then, streams and rivers such as Cherry Creek and the South Platte River flowed north; thus, cutting back down through the accumulation of sediments abutting the Front Range. Castlewood Canyon was created in this manner. The Castle Rock Conglomerate, capping the Dawson Arkose, provided a temporary barrier to the down-cutting process; thus, forming the wide valley upstream of the canyon head. Following erosion through the joint system, down cutting was resumed and the present configuration of Castlewood Canyon thus resulted. This restricted stream with a wide upstream valley would on the surface, appear to be an ideal location for a dam and reservoir. However, when the engineering properties of the Dawson Arkose and Castle Rock Conglomerate are taken into account, the location should have been bypassed, especially with the specific type of dam selected.

Dawson Arkose

Description – The Dawson Arkose portion of the D2 sequence is named from exposures in Dawson Butte, located south of Denver. This unit is characterized by alternating lenticular beds of arkosic sandstone dipping gently to the east from the Front Range. These multi-storied, river-channel arkosic sandstones are separated by over-bank mudstones. Weakly developed paleosols can also occur within the over-bank mudstone deposits. The Dawson Arkose generally overlies the Denver Formation; however these two units inter-finger in the Littleton area. In other areas, a significant thickness of the arkosic strata overlies the basal Arapahoe conglomerates with little or no andesitic sediments of the Denver Formation being present (Raynolds, 2002). Castlewood Canyon displays good exposures of the Dawson Arkose sequences, mostly as a result of the dam failure (Figure 2).

Figure 2. Right abutment slopes downstream of dam, showing excessive erosion in Dawson Arkose during and following failure. Note near vertical slopes in sandstone and less steep slopes in center mudstone

Origin – The composition of the Dawson Arkose (D2) suggests that it is a product of the redistribution of a grus mantle derived through uplifting of the Front Range Precambrian granites of the Pikes Peak area and eastward thrusting of the Ute Pass fault. One of the keys to this origin is the inclusion of small amounts of the rare mineral Amazonite (a sky blue feldspar) in both the Dawson Arkose and in the granitic rocks of the Precambrian Pikes Peak Batholith (Raynolds, 2002). An alternate explanation for the deposition of the Dawson Arkose could involve eastward tilting in response to a period of westward thrusting on the Elkhorn Thrust (Kluth and Nelson, 1988).

The Dawson Arkose (D2) displays more uniformity with less lateral compositional and textural variability than the lower Denver and Arapahoe Formations (D1). The observed thickness relations, together with northerly paleocurrent vectors observed south of Denver, suggest a source in the Pikes Peak area west of Colorado Springs. Contemporaneous subsidence of the Denver Basin generated the depositional space needed to contain and preserve the Dawson Arkose strata. Also, because the subsidence of the Denver Basin was not focused in a narrow zone parallel to the Golden and Rampart Range Faults at the eastern edge of the Front Range, the course fluvial sediments of the Dawson Arkose were carried beyond the mountain front source area (Raynolds, 2002).

Sandstone Engineering Properties – To call the Dawson Arkose a rock is quite charitable and was possibly the first mistake made by the designers of Castlewood Dam. Upon close inspection, the engineering properties, except for the long standup time, are those of a soil with minimal clay cementation and the dam designer should have treated this material as such.

The alluvial sandstone facies of the Dawson Arkose at the damsite are typical of those derived from a grus. Grain sizes are of fine sand with a trace of medium to coarse sand and containing much clay and silt; some of which is in the form of clay or mud balls the same size as the sand grains, with interstitial clay providing very poor cementation. The fines content is over 30% and determines this material's engineering characteristics. The Plasticity Index (PI) of this material ranges from 70 to 80 with very low shear strength; especially when wet (Terry, 2003). In cross section, the beds over long distances are lenticular; but at the dam this is not obvious and the bedding appears massive with no discernable bedding planes. When dry, this material behaves like a poorly cemented sand with none to low dry strength and low toughness; but when wet, it looses all strength and behaves like a clay or silt with very low plasticity. When dry, the standup times are very long; the eroded rock sequence at the damsite has vertical walls that have stood for nearly 70 years with only slight deterioration. However, when wet it looses all strength and slumps to low angle slopes. At the site, this sandstone performs much like a loess with long standup times when dry to moist and no strength when wetted. The sandstone is highly micaceous, dry to moist, light gray, unweathered, soft, weakly cemented and in the Unified Soils Classification System (USCS) would be classified as a silty, clayey sand (SC-SM).

Mudstone Engineering Properties – The alluvial mudstone component of the Dawson Arkose displays similar engineering properties similar to the sandstones; but with lower strengths. The material is composed of particles generally less than the 200 sieve size with small amounts of fine, micaceous sand. The material is uncemented with PIs in the 70 to 80 range and very low shear strengths (Terry, 2003). The material appears to be laminated. Over long distances, the beds are lenticular; but appear contiguous in cross section at the dam, with a total single-bed thickness of approximately 10 ft (3 m). The mudstones were derived from over-bank deposits during deposition of the sandstones (Raynolds, 2002). When dry to moist, the mudstones will stand for long periods at roughly 60-70 degrees; but when wet they exhibit no shear strength and will literally flow, as evidenced by mud streaking on the lower sandstones, and slump. The in-situ mudstones are moist but deteriorate and slake upon drying. At the damsite, the mudstones are unweathered with no evident paleosols, have very low dry-strengths, are non-plastic, are weakly cemented, are light to medium gray, and are very soft to soft. In the USCS, these materials would classify as sandy silt (ML).

Castle Rock Rhyolite

General - Although the Castle Rock Rhyolite does not play a part in the construction of Castlewood Dam, it is quarried in the immediate area. The rhyolite was deposited between the Dawson Arkose and the Castle Rock Conglomerate, and it supplies a percentage of the clasts in the overlying Castle Rock Conglomerate. Therefore, some description of its origin and properties is appropriate.

Origins – Approximately 36.7 mya, the Rocky Mountains had been leveled by erosion and the landscape of the Front Range was a gentle east-dipping pediment. At that time, a major volcanic eruption, presumably in the Sawatch Range, resulted in the deposition of an extensive welded tuff across the southern portion of the Denver Basin. Today the Castle Rock Rhyolite represents the deposits of this violent eruption. In areas the welded tuff is 20 ft (6 m), or more, thick and is present on top of buttes between Castle Rock and Monument Hill (Johnson and Raynolds, 2002). The Castle Rock Rhyolite occurs in Castlewood Canyon as clasts in the younger Castle Rock Conglomerate ranging from gravel to boulder size. Quarries in the Castle Rock Rhyolite are located on the second ridge west of Castlewood Canyon. The quarried and dressed stones from these and other quarries have been used in the past as architectural stone in many buildings along the Front Range.

Castle Rock Conglomerate

Description – The Castle Rock Conglomerate, Eocene, is found in an elongate-shaped series of outcrops 6- to 15-mi (10 to 25 km) wide, extending for 47 mi (75 km) from northwest of Castle Rock towards the southeast, near Calhan, Colorado (Morse, 1985).

The Castle Rock Conglomerate is well cemented and forms cliffs by capping the poorly cemented Dawson Arkose (D1) and other units in eastern Douglas, southwestern Elbert and northern El Paso Counties. This conglomerate consists of very coarse conglomerate with interbeds of arkosic sandstone and minor beds of massive olive-green mudstones. Large-scale crossbedding is common. The maximum thickness of conglomerate is approximately 230 ft (70 m) just east of Castle Rock. The type section is the butte known as "Castle Rock" in the town of Castle Rock. The Castle Rock Conglomerate overlies both the Castle Rock Rhyolite and the Dawson Arkose (Morse, 1985). In Castlewood Canyon where the conglomerate lies directly on the Dawson Arkose, large cobbles and boulders of subangular to subrounded rhyolite are included, indicating a source and contact with the rhyolite within a short distance to the west. At the Castlewood Dam site, the conglomerate lies directly on an uneven surface cut into the Dawson Arkose. In addition to the large Rhyolite cobbles, boulders and large blocks (some of which are elongate), the conglomerate contains cobbles and boulders of granite, quartzite and gneiss (Morse, 1985). The particle sizes of the conglomerate range from finer than the 200-sieve size up to boulder size. In the upper portion of the conglomerate, which was used in the construction of Castlewood Dam, clast sizes are mostly in the gravel to cobble ranges.

Origins – The Castle Rock Conglomerate is interpreted to be the deposit of a moderately deep, braided, gravelly stream of high energy, low sinuosity in high relief valleys. Flood depths were at least 10 to 16 ft (3 to 5 m) and locally, gravelly point bars may have been present, although deposition was primarily by mid-channel bars. The deposition of the Castle Rock Conglomerate is analogous to present-day depositions of the South Platte River to the northwest (Morse, 1985).

Discontinuities – One, well-developed, primary joint set is evident in the conglomerate on the right abutment, bearing approximately N10W (Figure 3). Joint spacing varies from 2 to 8 ft (.6 to 2.5 m). This well developed joint set is nearly vertical and was used in the quarries to obtain uniform blocks for dam construction. A second, less well-developed joint set is evident at right angles to the primary set, also with a vertical dip. On the upper left abutment, joint orientation is approximately the same as on the right abutment. Evidence of bedding planes or horizontal joints exists in the quarry on the right abutment. Some curvilinear joints are also present in the quarry on the right abutment.

Figure 3. Right abutment quarry displaying primary joint set in conglomerate. Note irregular rock breakage in center of photo.

Engineering Properties – The Castle Rock Conglomerate is well cemented, durable, and is of high compressive strength; which made it suitable for obtaining rock-blocks for the construction of Castlewood Dam (Figure 4). However interbeds of less well cemented sandstones and mudstones degrade the quality of the rock in areas. The well-defined joint sets allow the quarrying of somewhat uniform sized blocks suitable for this type of construction. However, adverse curvilinear joints, weak bedding planes and other minor adverse joint orientations result in a high number of irregular blocks, making fitting, placing and mortaring of the blocks very difficult. Much evidence of the poorly shaped blocks can be seen in the dam, resulting in less than ideal structural and water retention integrity of the upstream wall and downstream stair-stepped shell. In addition, the conglomerate is not uniformly cemented or bedded which results in much fragmentation. This high degree of fragmentation is evident in the rubble fill and resulted in most of the irregular rock being used in the rubble fill. The Castlewood Conglomerate is rarely used as a building stone in the local area. On the other hand, the Castle Rock Rhyolite in the area near the dam is much better suited for shaping.

Figure 4. Conglomerate block, stair-stepped downstream face. Note irregularities of rock surfaces and deterioration of interstitial mortar.

DESIGN AND CONSTRUCTION

Design

Castlewood Dam was a maximum of 70-ft (21m) high with the upstream wall 92 ft (28 m) above the foundation, 50-ft (15 m) wide at the base, 600-ft (183 m) long, had a crest width of 8 ft (2.4m), with a crest elevation of approximately 6450 ft (1976.3 m) (Figure 5). The reservoir

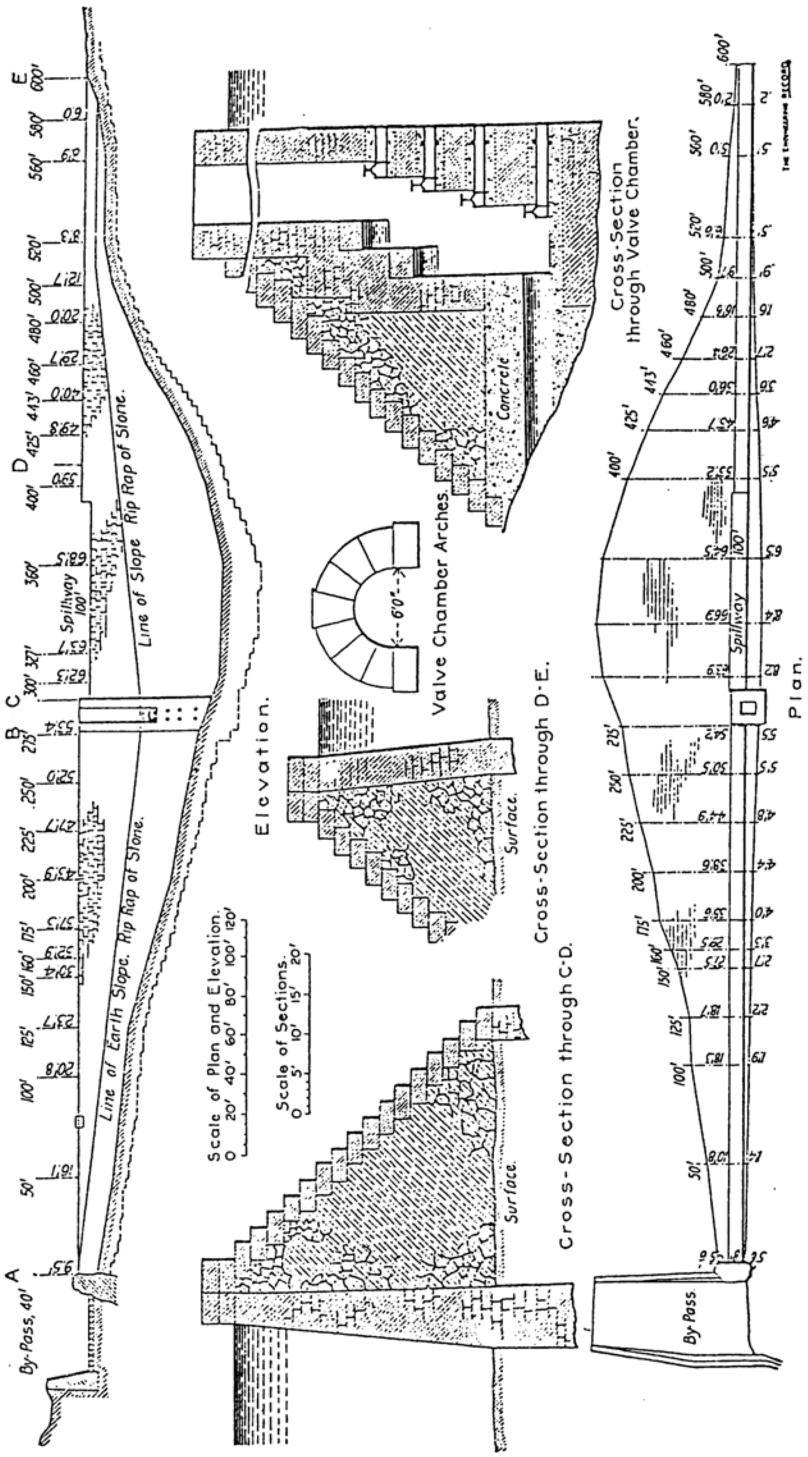

Figure 5. Castlewood Dam; Plans, Sections, and Elevation (Schuyler, 1909).

collected water from approximately 175- to 200-square mi (45,300 to 51,800 hectares) of the upper Cherry Creek drainage basin and had a designed capacity of 12,280 acre-ft (15,147,000 cu m) (Schuyler, 1909). Current studies have reduced that capacity to approximately 5,000 acre-ft

(6,165,000 cubic m) (Graham, undated). The water from the reservoir was intended to irrigate approximately 16,000 acres (6,480 hectares) between Castlewood Canyon and Denver (Horan, 1997). Little is known about the intent or particulars of the design of Castlewood Dam and we must look at the completed structure to obtain insight into its design; however, the various accounts and particulars of the constructed dam conflict and thus, the "best fit" particulars were used.

Construction

Castlewood Dam was a masonry rubble-fill structure with a water-retention, masonry wall on the upstream face. The downstream shell was stepped with large mortared blocks of dressed conglomerate in approximate 2 ft (.6 m) steps having a general batter of 1:1 and placed directly on the loose foundation rock. The upstream masonry wall was constructed of dressed conglomerate blocks set in mortar, with a base width of 11 ft (3.4 m) and 5 ft (1.5 m) at the top, and having an overall batter of 1:10. The upstream masonry wall and the downstream stair-stepped shell met 12 ft (3.7 m) below the crest and formed a solid, masonry wall upward. The crest of the dam was capped with large quarried blocks of conglomerate with a total width of 8 ft (2.4 m) [2 blocks wide] and a depth of 4 ft (1.2 m) [1 block thick]. The rubble fill consisted of large conglomerate stones laid in-place by hand with the voids being filled by broken rock, hammered into place. According to plans, the upstream wall was founded from 6 to 22 ft (1.8 to 6.7 m) below the top of ground and the downstream facing was founded 10 ft (3 m) below ground surface (Schuyler, 1909, USGS, 1899). The current configuration does not bear this out.

The main spillway consisted of an ungated notch in the center, top of the dam. This notch was 100-ft (30.5 m) long and 4-ft (1.2 m) deep [one block]. The emergency spillway (called the by-pass spillway on the drawings) was an uncontrolled structure located at the extreme left end of the dam and was 30- to 40-ft (9.1 to 12.2 m) wide. The total combined discharge volume of the two spillways was 4,000 cubic ft/sec (113 cubic m/s); which with another 250 cubic ft/sec (7.0 cubic m/s) from the outlet works gives a total outflow capacity of 4,250 cubic ft/sec (120 cubic m/s) (Schuyler, 1909). In contrast, a potential inflow volume from Cherry Creek could be 10,000 cubic ft/sec (283 cubic m/s) for short durations during summer thunderstorm events (Graham, undated).

The control tower was a rectangular structure, built into the dam with an opening of 6 by 7.5 ft (1.8 by 2.3 m) at the top. Inside this tower, eight each, 12-in (305 mm) diameter pipes discharged at 4 successive levels starting at intervals of 6 ft (1.8 m) above the base. The outlet consisted of a 36-in (910 mm) diameter concrete pipe encased in 4 ft (1.2 m) of concrete. The outlet emptied directly into the stream channel and was diverted into irrigation ditches at the mouth of Castlewood Canyon, 1.5 mi (2.5 km) below the dam, through a 140-ft (43 m) long, low diversion dam. Dam construction was started in December, 1889 and completed in November 1890.

Design and construction supervision was provided by A.M. Welles, CE of Denver with consulting engineering services provided by Alred P. Boller, M. Am. Soc. CE of New York (Schuyler, 1909, USGS, 1899, Appleby, 2001).

The designer felt that the rubble fill dam would be safer than a masonry dam; because the rubble fill would stay in place if the dam completely failed (Schuyler, 1909).

Reconstruction

During and following construction, the residents downstream continually questioned the safety of the structure. In the mid 1890s, the dam was inspected by the original designer and it was determined that construction was faulty (Figure 6). However, the designer felt that the dam could be made safe by the addition upstream of an extensive earthen berm, constructed to the top of the spillway and riprapped. A partial berm had already been installed during construction; but was not completed as intended by the design/construction engineer. This recommendation to complete the upstream berm was not carried out until the dam partially failed on August 3, 1897 as a result of heavy rains in the area (USGS, 1899).

Figure 6. Completed dam prior to reconstruction with extensive seepage through dam on left abutment. Note riprap placed along the downstream, right abutment groin (Colorado Historical Society).

This partial failure was caused by wave action undermining the upstream masonry wall on the right abutment. The damage consisted of settlement and several cracks 2-to 4-ft (.6- to 1.2-m) deep and extending horizontally. Water poured through the cracks and exited on the lower face of the dam. The reservoir was drained; but filled suddenly the following year during a large thunderstorm. At this time, water passed through the cracks without further damage to the upstream wall. However, upon exiting the downstream toe, the water ran down the groin, undermined the toe of the downstream stepped shell, causing it to settle and bulge outward. At the west end of this failure, a large crack ran up the natural slope (USGS, 1899).

During the first eight years of operation, another comparatively slight settlement occurred to the left end of the upstream masonry wall. The settlement occurred 125 to 150 ft (38 to 46 m) from the left end of the dam, where the lower half of the 8-ft (2.4-m) wide crest, resting on loose rock, settled. A large crack rose to the crest and extended nearly to the end of the dam. Superficial repairs were made by raising the lower portion of the wall and filling the crack with chinked mortar (USGS, 1899). However, no repairs were made to the foundation, and a void was probably left.

In1898, repairs were started to the masonry wall on the right abutment by removing the upper portion of the wall resulting in an excavation approximately 30-ft (9 m) long. The base of the excavation was widened and backfilled with a concrete footing. At the toe of the wall where original failure occurred, a footwall was constructed and the original wall was restored (USGS, 1899).

At the same time, an apron was constructed at the toe of the dam in the area of the spillway to dissipate the energy from spillway discharges and to prevent erosion of the toe during spillway use. This apron was carried 50 ft (15 m) beyond each end of the spillway, for a total length of 200 ft (61m) and was constructed of pavement stone. Reportedly the apron-stone pavement was grouted at the top with heavy mortar. This apron was 25-ft (7.6m) wide at the base (USGS, 1899). No evidence of this apron remains.

During reconstruction of the dam, an upstream berm was constructed to reduce the seepage flows through the upstream masonry wall. This upstream berm was constructed to within 35 ft (10.5 m) of the top of the dam at the outlet-works near the center and was raised to the full height of the dam at either extremity. The slope of this upstream berm was 3 horizontal to 1 vertical (Schuyler, 1909). The type and source of the material used in this berm is unknown.

During construction of this upstream berm, it was strengthened with a puddled-clay wall placed directly against the upstream, masonry wall. The puddled-clay wall was 3-ft (1 m) wide at the top of the left abutment, increasing to 8-ft (2.4 m) wide at the valve chamber. The puddled clay was not carried across the 150 ft (46 m) valley floor; but was carried up the right abutment with a width of 6 ft. The puddled-clay wall was extended downward to the top of original ground for its entire length, except where the original berm was left in-place, along the repaired break in the dam, on the right abutment. At this location, the puddled-clay wall was carried down to the bottom of the repair. At either end of the dam, the puddled-clay wall was extended 1 to 2 ft (0.3 to 0.6 m) into the natural ground. Riprap of heavy stone, one-foot-thick, was placed over the entire berm to protect it from wave action (USGS, 1899). No sign of this riprap remains.

Shortly after the dam was completed, it began to leak excessively; especially through the left section of the dam. Within 6 months of completion, a small fissure developed along the crest of the dam that required patching. At one time before reconstruction, probably in 1897 following a heavy rain, the reservoir was filled, the dam was over topped and water poured out in large volumes through the downstream rock/rubble fill on the left side for 10 to 15 ft (3 to 4.6 m) above the base of the dam (Figure 7). However, no apparent damage was incurred to the dam structure (Schuyler, 1909). This issue continued to be a problem throughout the life of the dam.

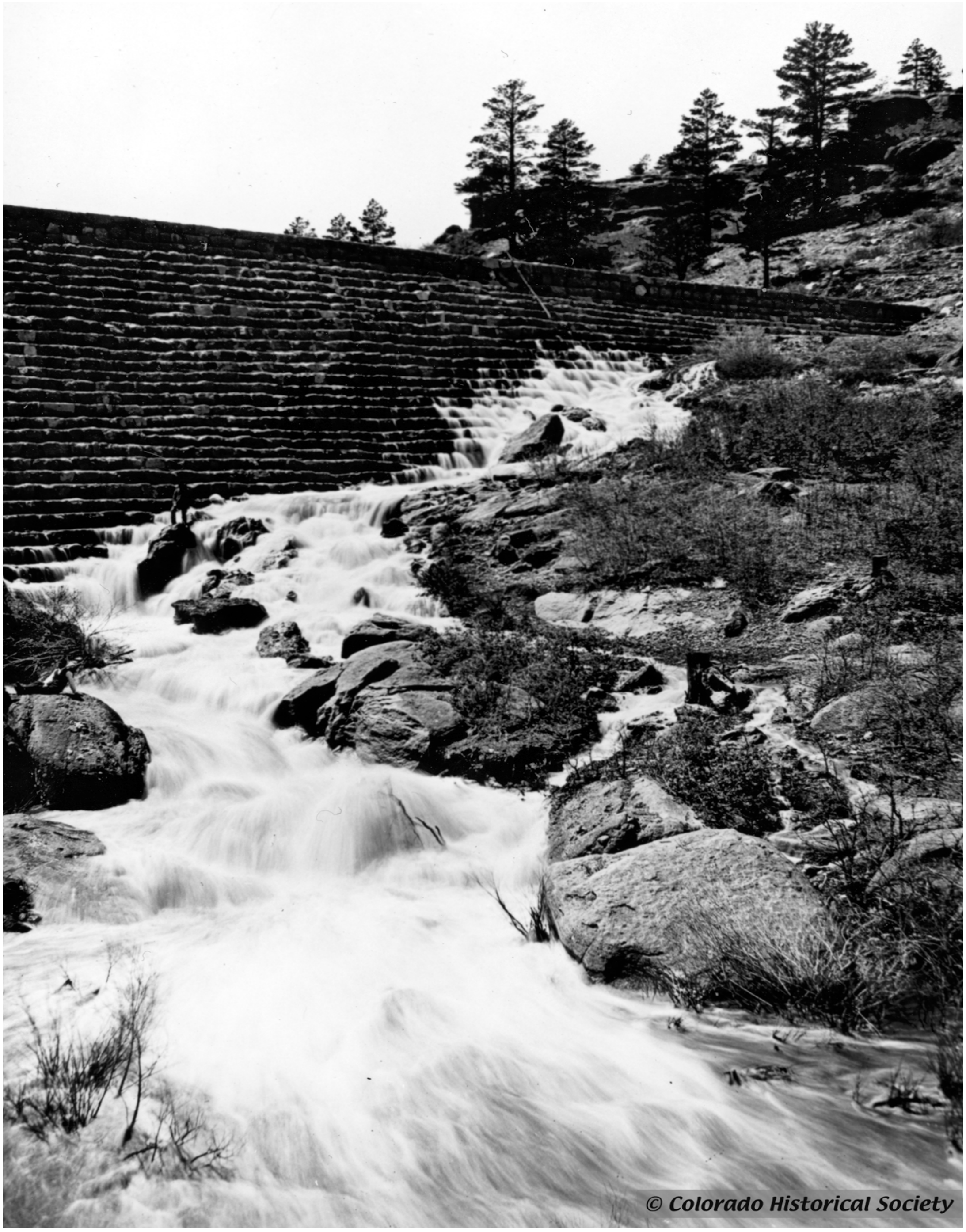

Figure 7. High amount of seepage through dam on left abutment, prior to reconstruction. Outlet pipe is just out of view to left of photo (Colorado Historical Society).

As part of the reconstruction, the intake structure was redesigned (Figure 8). A concrete intake tower and valve chamber was constructed against the upstream masonry wall to protect the intake valves. This tower was extended to the crest of the dam. In addition, an intake pipe was added beneath the earthen berm. This intake pipe consisted of a 40-in (1 m) diameter wood-stave pipe with metal reinforcing hoops, surrounded by concrete up to 1-ft (0.3 m) thick. Two concrete seepage collars were provided, each with 1-ft (0.3 m) thick and 7-ft-8-in (2.3 m) square dimensions (USGS, 1899, Schuyler, 1909).

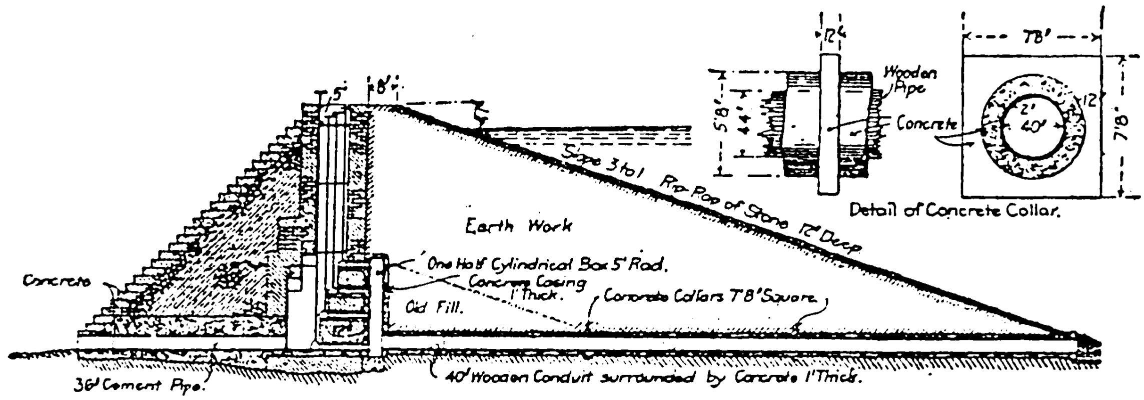

Figure 8. Castlewood Dam; cross section through intake/outlet works and upstream berm after reconstruction (Schuyler, 1909).

The unlined, 30- to 40-ft (9.1- to 12.2- m) wide spillway on the left abutment was seriously damaged during the 1897 overtopping event and was repaired to include constructing a masonry sill to 1 ft (0.3 m) above the crest of the dam and building a masonry wall to the left of the sill structure. This raised sill precluded use of the spillway except during extreme overtopping events (Figure 9). The spillway was left unlined downstream of the sill structure. Another spillway was constructed at the extreme right abutment. This auxiliary spillway did not have a control structure and was left unlined also. The right abutment spillway was 12-ft (3.7 m) wide with side slopes of 1:1 and was constructed to empty back into Cherry Creek approximately 100 ft (30 m) downstream of the dam at a level just below the crest elevation (Schuyler, 1909) with resulting free-fall of the discharge. Shortly after this reconstruction, the Denver Land and Water Storage Company went bankrupt in 1901. The reported extent of repairs and the dates of these repairs are inconsistent between the various references and the accounting in this paper reflects the best estimate of these activities and dates.

OPERATIONS AND FAILURE

Operations

For the next 30 years, Castlewood Dam was operated without newsworthy events; except, for the recurring inability to hold sufficient water to meet downstream demands and the various bankruptcies. Also, the debate continued in the Denver newspapers about the safety of the dam with a scathing reply by the designer/construction engineer in 1906. This apparently did not stop the debate and the question of the dam's safety continued right up to the time it failed in 1933. Following the bankruptcy of the Denver Land and Water Storage Company, the dam and reservoir passed through various land speculation ventures until finally it was tied up in court from 1912 over the rights to the stored water. In 1923, individual landowners began collecting the water rights and from 1930, until it failed, a group of 150 downstream irrigators managed the

Figure 9. Left spillway following reconstruction. Dam crest is at right of photo. The reconstructed spillway sill is at left of photo. Note large conglomerate boulder in center of photo that forms left end of dam. Also note left spillway retaining wall at far left side of spillway, erected during reconstruction.

dam and reservoir under the title of the Cherry Creek Mutual Irrigation Company. The water from Castlewood Reservoir was then used to irrigate approximately 2,500 acres (1,000 hectares), mostly alfalfa, grains and miscellaneous crops (Horan, 1997).

Failure

The summer of 1933 brought higher than normal rainfall to the Denver area. On the night of August 2, 1933 more than 8 in (200 mm) of rain fell in the Cherry Creek drainage basin upstream of Castlewood Dam in less than three hours. This thunderstorm was in addition to at least 3 previous days of heavy rain, which saturated the ground (Graham, undated).

At 11:45 pm on the night of August 2, 1933, the dam tender (Hugh Paine) observed the dam and the water was 6-ft (1.8 m) below the center spillway crest. By midnight, the water had risen over the spillway crest to the top of the dam, approximate elevation 6450 ft (1967.3 m). By 12:15 am the next day, a torrent of water was pouring over and through the dam and within a few minutes, the surface had dropped to 13-ft (4 m) below the center spillway crest (Graham, undated).
The lapsed time between the inrush of water into the reservoir and the time of failure probably did not exceed 45 minutes. During this time, the dam was overtopped its full length to a depth of at least 1 ft (0.3 m), with an estimated discharge of 3,000 cfs (85 cubic m/s) through the spillway notch, and an estimated overflow of between 2,500 and 3,000 cfs (71 to 85 cubic m/s); for a total flow over the dam of 5,500 to 6,000 cfs (156 to 170 cubic m/s). It is estimated that 2,500 acre-

feet (3,000,000 cubic m) were stored in the reservoir at the time of the thunderstorm and there were 5,000 acre-feet (6,200,000 cubic m) at the time of failure (Graham, undated).

Since it was night, little is known of the exact cause of the dam failure. The dam tender was the only observer and his comments only indicate that at the last observation, the dam was breaking up and that water was pouring over the top. Just before the failure, the dam tender was concerned that the valves were not opened and that he wanted to open them; but the dam was "rumbling" so badly that it was unsafe to climb down the 70 ft (21 m) control tower to accomplish this task (Horan, 1997).

Disaster was averted by quick actions of the dam tender and the persistence of many private phone company operators. At the time of the failure, at Castlewood Dam, the telephone lines were out and the dam tender rode the 12 mi (20 km) to the Castle Rock phone exchange to warn downstream residents of the impending flood. From there the warning was relayed through persistent telephone operators to the people downstream of Castlewood Canyon; often the operators stayed on the party lines until they were sure that all had been warned of the impending flood. Thus the loss of life was kept to only two persons; but the property damage reached upwards of 1 million dollars (Horan, 1997).

CURRENT CONDITIONS

Right Side

The entire right side of the dam is missing except for a very small section at the extreme right abutment. On the right side where the dam has disappeared, there are no signs of extensive erosion and the only signs that a dam was ever there are slightly steepened slopes, upstream and downstream of the missing dam. On the upper right abutment, no erosion is evident, except in the area of the right abutment spillway. The question is how did the entire right side disappear with little trace of ground erosion? Photos taken shortly after the dam failed indicate complete failure of the right side with little erosion occurring in that location, but with some deposition.

Center Section

In the center section of the dam, just to the right of the outlet works where the spillway opening once existed, the dam is entirely gone. Also, in this area, extensive erosion of the foundation has occurred. This erosion and entrenchment of Cherry Creek is approximately 20-ft (6 m) deep below the base of the original dam. However, much of this erosion has taken place in the years since the dam failed and it is not possible to tell exactly how much erosion was caused as the dam failed and how much occurred since the dam failure. However based on the depth of cut, the extent of headward erosion and the base of the rubble pile above the stream channel; it appears that most of the erosion has taken place since dam failure and particularly in 1937 during periods of heavy rains.

The intake tower and outlet works are nearly intact (Figure 10). Some of the outlet pipe is missing downstream. However, the outlet pipe still exists in the approximate footprint of the original dam. Two possible reasons it remains are: (1) a large conglomerate block exists at this location and it is clear that the dam and outlet were constructed around this large rock-block; and (2) the outlet works was encased in concrete giving it added strength within the dam footprint. The outlet-works tower, with a triangular cross section, is the only portion of the dam constructed with large amounts of concrete and consequently is stiffer and better able to resist erosion. The concrete intake tower upstream of the dam, part of the reconstruction, was formed for its entire height except for that portion which was below original ground (approximately 6 ft [2 m]). Below ground level, the concrete surfaces were placed directly against the Dawson Arkose and when that rock was washed away; an irregular textured surface to the exposed concrete was left. This intake tower construction was probably doweled into the masonry upstream wall of the dam, as evidenced by the lack of separation

Figure 10. View across valley at remaining right side of dam, center right of photo. Note upstream control-tower extension and upstream berm.

The intake pipe upstream of the dam is partially intact. The wooden staves have long since vanished, but the metal hoops remain mostly intact. The intake pipe was also encased in concrete and this in combination with its upstream location, away from the severest erosion, allowed most of this pipe to resist the erosional forces created during failure.

The upstream masonry wall of quarried conglomerate is intact at the intake- and outlet- works area and a large triangular, well-mortared series of blocks remains in the dam core area downstream of the upstream masonry wall and directly above the outlet works (Figure 11 and 12).

Figure 11. Remaining portion of dam from control-tower to left abutment. Note condition of rubble fill and downstream stair-stepped masonry shell, 1971 (Colorado Historical Society).

The gate chamber and valves probably still exist, but are inaccessible. Still there is an open connection between the top of the gate tower and the downstream end of the outlet pipe. Since there were no gates in this structure, just valves, they probably remain closed, as on the night of August 2-3, 1933.


Figure 12. Close-up of remaining rubble fill and stair-stepped downstream, rock-block shell. Note poor condition and chinking of mortar between conglomerate blocks.

Left Portion of the Dam

For a distance of approximately 20 ft (6 m) to the left of the triangular outlet-works section, most of the rubble portion of the dam and downstream stair-stepped masonry shell has have been removed by erosion, leaving a small portion at the base of the dam. Beyond that, a portion of the rubble fill has been removed under the downstream stair-stepped masonry shell, leaving the underside of the shell hanging and open for inspection. Here, chinking of the mortar between blocks is very evident. Also in this area, a piece of pine bark was found embedded in the interstitial mortar. Further in this general area, a large pine log was found in-place within the rubble fill. Observing the interior of the dam between the upstream masonry wall and the downstream masonry shell, the rubble fill is totally uncemented and currently contains large voids. It thus appears to have been dumped instead of hand-placed and chinked as reported in the construction synopsis. Also in areas open to inspection, a portion of the rubble was in-filled with sand. This loose packing of the rubble fill would have led to the high permeability exhibited once the water managed to penetrate the upstream masonry wall. A 1971 photograph of the disrupted portion of the dam immediately to the left of the control tower structure shows a poorly segregated rubble fill. The fines have since disappeared. Leaving only large fragments and voids. This would further indicate a dumped fill without proper gradation or compaction.

Upstream Masonry Wall

From just right of the outlet-works, all the way to the left auxiliary spillway, the upstream masonry wall, the main and only water barrier structure, is still intact (Figure 13). Also in the area just to the left of the outlet-works where the rubble fill and the downstream shell have been removed, the construction and condition of the upstream masonry wall can be observed; or more specifically, the condition of the downstream portion of the wall can be observed. This downstream portion of the masonry wall was not constructed with the same integrity or care as the upstream portion of the wall. In this downstream section of the wall, the individual rock-blocks are irregularly shaped and the spaces between the blocks were mortared in a manner similar to that of the downstream shell. The mortar was chinked wherever possible to keep the amount of cement to the very minimum. In addition, there are numerous voids between the mortar and the rock blocks. Many of the voids are up to 2- to 6-in (50 to 155 mm) wide. In the process of placing masonry, normal practice is to place a thick bed of mortar on the top of the lower block to insure that when the upper block is set in place, complete contact is made between the two blocks. This practice was not followed and by adding the chinking, point-to-point, rather than full contact, is evident. It also appears that the mortar was placed wet, so that upon curing, it shrank away from the block it was supposed to be sealing; or voids were knowingly left between the blocks. Either way, this construction lapse was not conducive to constructing a water retention structure, especially when the water barrier was so thin to begin with. The upstream course of rock-blocks can only be seen from the upstream side of the dam and most are covered with the soil berm placed during reconstruction. Of the upstream blocks that can be observed, it is clear that they were patched after construction. Two distinct types of mortar are evident and the post construction mortar is smeared on the surfaces of the wall.

Just to the right of the control structure, a small portion of the upstream masonry wall remains and is cantilevered out over the stream channel, which was eroded as the dam failed. Since this is the lowest portion of the dam, inspection can be made of the construction methods at the foundation contact and the foundation treatment. At this point, the foundation construction consisted solely of a leveling-pad to give an even surface upon which to place the first course of rock-blocks. There is a high percentage of cobble size chinking material in this leveling-pad. The thickness of the leveling-pad is less than 3 ft (1 m) and it does not appear that there was any intention of tying this leveling pad to the upstream-downstream sidewalls of the excavated rock. The leveling pad sides are smooth, not irregular as would be expected if placed directly against the rock. Also, the bottom surface appears to be smooth.

Figure 13. Exposed downstream face of upstream masonry wall. A small portion of downstream shell is evident in lower right of photo

Crest

Viewed from the crest of the dam, there appears to be a separation between the two rows of quarried-blocks, indicating possible downstream movement of the downstream row (Figure 14). This is probably the first required repair to the dam mentioned in the construction section. Also, it is obvious that this original repair did not last; since further downstream separation has occurred over time. Also, when looking at the crest of the dam, much mortar at the surface is missing between the two main rock-block rows, possibly exaggerating the amount of observed separation.

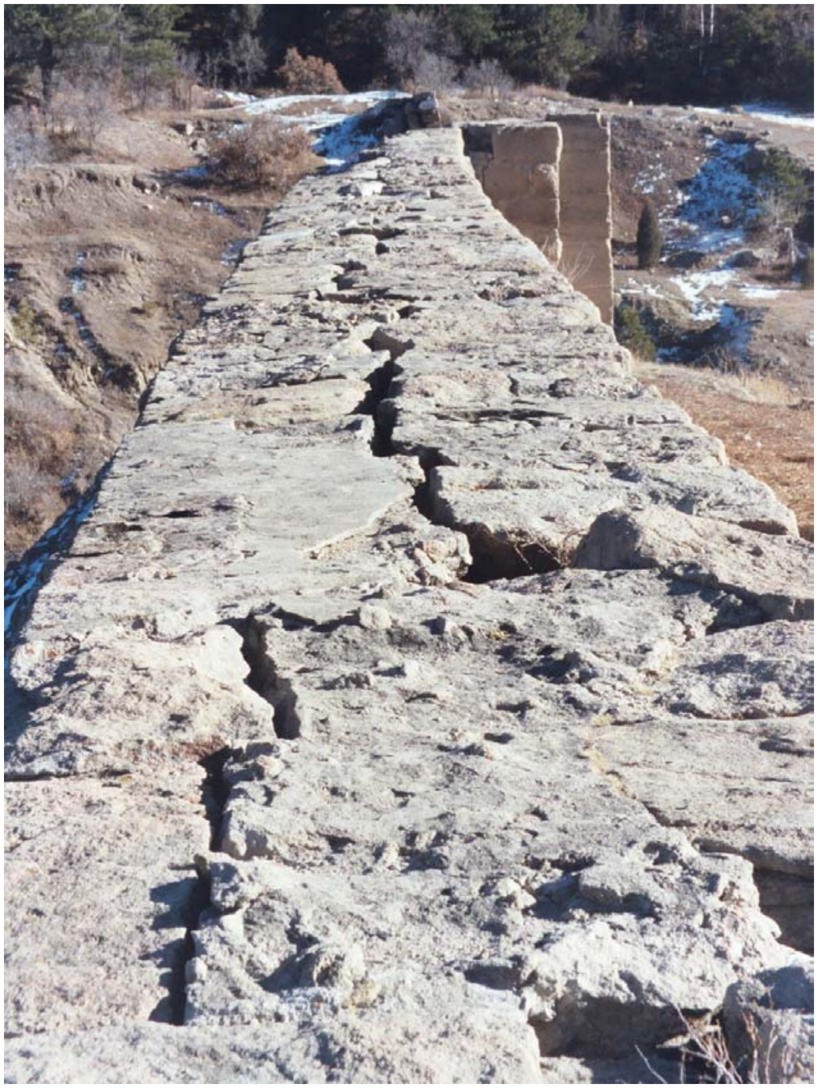

Figure 14. Left crest of dam. Note *minor* longitudinal crack.

Left Abutment Spillway

At the extreme left abutment, the dam is tied into a large, house-sized, conglomerate boulder left in place. The original auxiliary spillway was constructed as a trench to the left of this large in-place boulder, which also formed the left end of the dam. It appears that the original spillway in this location was not lined and was not protected by either a sill structure or retaining walls. The current sill structure, constructed as part of the 1898 reconstruction, is constructed of a one-layer thick, mortared conglomerate-block wall. The conglomerate blocks average about 1-ft (0.3 m) thick and are considerably smaller than the blocks used in the main portion of the dam. The sill structure was clearly constructed in one operation, with no obvious break or change in color as would be expected if raised at a subsequent time. The upstream sill continues to the left forming the left retaining wall of the spillway, which in turn is abutted to natural material of the Dawson Arkose (Figure 9). The Castle Rock Conglomerate contact is much further up the hill, above the dam on the left abutment. The floor of the spillway is unlined.

The spillway discharge-channel follows an apparent natural draw down the side of the valley and exits into the main channel approximately 100 ft (30 m) downstream of the downstream toe of the dam and approximately 20 to 30 ft (6 to 9 m) above the current streambed (Figure 15). It appears that when the spillway was first used, a great deal of erosion occurred in the discharge-channel, which subsequently endangered the downstream toe of the dam. The extensive damage caused by the early flows appears to have been subdued over time. Currently, the eroded walls are somewhat rounded and only minor secondary erosion is evident with occasional large, exposed conglomerate boulders. With construction of the present spillway sill about 1 ft (0.3 m) above the crest of the dam, it was clearly intended that this spillway would only be used in emergencies, after the dam had already overtopped.

Figure 15. View of spillway discharge-channel on left abutment. Downstream toe of dam is just to left of the photo. Note large pile of dam debris at downstream toe of discharge-channel, along lower portion of photo.

Downstream Area

At the downstream end of the left spillway discharge-channel there is a huge pile if dam debris, consisting of all sizes of blocks from the dam along with portions of the rubble fill. The blocks are squared and some display the characteristic shot holes used to break the rock in the quarry. The placement is chaotic, suggesting that as the dam failed, this mass of rubble was rotated to the left and dropped in its present position. The base of this rubble pile is approximately 20 to 30 ft (6 to 9 m) above the current stream channel and above the lowest terrace; indicating that most of the erosion in the current stream channel has occurred since the dam failed in 1933. Occasional large squared blocks of conglomerate can be found downstream, up to ¼-mi (0.4 km) below the dam. Some of these blocks display the characteristic drilled shot-holes. At this point, the stream channel widens and flattens, making tracking of the dressed blocks very difficult. There are undoubtedly blocks further downstream, as the volume of the dam was much greater than could be accounted for in the quarter-mile-stretch downstream of the dam.

Subsequent Erosion

In the valley section of the dam, tremendous erosion has taken place and the current streambed is approximately 20 ft (6 m) below the base of the original dam. Not all of this erosion was a direct result of the dam failure and most has occurred over the 70 years since the dam failed. The exact amount of erosion that occurred at dam failure cannot be precisely determined, which makes failure determinations extremely difficult. Currently, the walls of the present stream channel are nearly vertical with slightly less steep slopes (approximately 60 to 70 degrees) in the mudstone bed at the very base of the dam. Surprisingly, incising of the stream channel has not progressed upstream more than a-third-of-a-mile (1/2 km).

Upstream Berm

The upstream earthen berm, which was placed against the upstream face of the dam in an attempt to reduce seepage through the dam, is still in place to the left of the outlet-works (Figure 16). The exact method of placement of the berm is not known and the exact source of the material used in this berm is unknown. Evidence of the puddled-clay portion could not be observed due to a graveled walkway constructed in this area. The upstream slope of the berm is steeper than 3:1 and it contains much dark-brown topsoil within the fill. It was reported that the berm was

Figure 16. Remaining upstream berm on left abutment. Note remains of intake pipe at center of photo.

placed to the top of the crest (Schuyler, 1909); but currently, the top of the berm is over 4 ft (1.2 m) below the crest elevation, attesting to a large amount of settlement and questionable compaction methods. This berm also restricts visual observation of the upstream face of the dam.

DISSCUSSION

General

Many of the items described in written documents concerning the design and construction of Castlewood Dam are not evident at the site and most of the discrepancies would not have been removed during the failure. Also, some of the basic dimensions quoted in publications are either in dispute or vary depending upon the source. For instance, the capacity of the reservoir is variously quoted as being anywhere from 5,000 to 12,000 acre-ft (6,200,000 to 14,800,000 cubic m). The capacity may have been exaggerated to sell irrigation water downstream.

Due to the darkness, there were no witnesses to the dam failure and the only first hand knowledge we have about the dam failure comes from the dam tender. Just before the dam failed, the dam tender was concerned that the valves were still closed; however the dam was already starting to break up and it was unsafe to make any attempt to climb down the control tower to open the valves. The effect of the valves being closed was minimal and probably did not contribute to the dam failure. Also, the water level reportedly dropped approximately 13 ft (4 m) just before the dam tender left to warn others and this indicates that the dam had essentially failed by this time. The dam tender made no other recorded observations concerning the dam failure.

The Character of the Foundation

There is a tremendous difference between the slopes in the Dawson Arkose at the site of Castlewood Dam. In the current stream channel, the slopes are nearly vertical and these near vertical slopes have been maintained over the last 70 years since the dam failed in 1933. On the other hand, the slopes in the Dawson Arkose higher up the abutments in the areas where the dam failed on the right abutment and in the old spillway channel on the left abutment are less steep, more rounded and more mature. Thus, giving the impression that these slopes have healed and softened over time. This healing and softening could give the false appearance of stability. We know that when the shallow left spillway was first used, concern was expressed over the great amount of erosion in the discharge-channel, jeopardizing the downstream toe and foundation of the dam (USGS, 1899). As a result of this heightened concern, a sill was constructed higher than the dam crest in effect making that spillway unusable. At the same time, another spillway was excavated on the right abutment to a configuration similar to the original spillway. It seems that little was learned about the character of the foundation.

The nearly vertical slopes in the present stream channel are being maintained by continued erosion, caused by periodic stream flows. Apparently in the absence of periodic high volume flows, the oversteepened slopes are not maintainable for lengthy periods and, due to the high clay content, small amounts of water may cause collapse of the interstitial clay balls; resulting, in the appearance of more rounded, gentle, or healed slopes once the original erosion scars have been erased. In the case of the left abutment spillway, the discharge-channel is now very pronounced, but the side slopes give only the appearance of having been very slightly eroded during the original discharges.

The same apparent stability is evident on the right abutment where the main dam failure occurred. These eroded slopes have been rounded and appear today as having been only slightly eroded during the dam failure. This illusion is made more complete by the appearance of two apparent stream terraces high up on the right abutment. When first observed, these two apparent terraces appear to predate the dam and one appears to line up very well with the oldest terrace upstream, which possibly predates the dam (Figure 17). However due to the method of dam construction, these two terraces should not have predated the dam. Appearances can be deceiving and this may have been one of the items that encouraged the designers to build Castlewood Dam at the chosen location.

Figure 17. Terraces in valley upstream of dam. The lowest terrace, center of photo, may also pre-date dam failure. Note excessive erosion following dam failure

This false ability to heal itself, somewhat typical of a grus, may also account for the dam's ability to maintain itself on the left abutment where extensive leakage occurred immediately following construction. Normally, given this type of highly erosive material and the large amount of seepage on the left abutment, immediate damage and settlement of the downstream face, if not complete failure, could have been expected. This did not occur and the dam was able to withstand the internal erosion forces for almost 43 years. Apparently the loose rubble fill protected the Dawson Arkose foundation from extensive and rapid erosion and let the rock/soil readjust without failing

The upstream terraces are of little help in unraveling the amount of erosion directly caused by the dam failure. Of the approximate 6 terrace levels, most were probably created during reservoir operations. The lowest terrace within the river channel appears to predate the dam due to the mature cottonwood trees growing on it, which are less than 70 years old, and a rudimentary soil horizon. The most severe erosion is below this terrace and there is no clear-cut distinction between failure and post-failure flood events. The ability of the sandstone to maintain vertical slopes for long periods of time further complicates the determination of distinct flood events.

Photographs taken shortly after the failure seem to indicate that erosion at the time of the failure was limited vertically to the base of the dam or very slightly below and that there was no deep gullying as is now present. The quality of the photos is not great; so, this determination is also not clear.

Selection of Facing Stones

The use of the local Castle Rock Conglomerate for facing stones was not the best of choices (Figure 18). Upon first observation of the conglomerate, the primary and secondary joint systems seem favorable for producing rectangular rock-blocks with uniform surfaces. However,

Figure 18. Upstream face at crest of dam. Note irregularity of conglomerate rock-blocks and very poor condition of mortar with multiple patching.

closer inspection of the quarry and the facing stone used in the dam reveal other minor joint sets and flaws, which together with the irregular clast sizes disrupt the regularity and adherence capabilities of the block surfaces. This in turn leads to irregular and uneven rock-blocks that were less than ideal for the type of construction that depends upon the regularity and uniformity of the individual blocks to maintain structural integrity. Although not a critical flaw, the use of irregularly shaped rock-blocks combined with the use of low quality mortar resulted in excessive leakage and some settlement and redistribution of the blocks within the downstream face. Although silicate cemented and generally durable, the conglomerate blocks have exhibited some weathering degradation since their installation in 1890.

A better choice for facing stone would have been the Castle Rock Rhyolite, which was available less than 3 mi (5 km) away on the second ridge to the west of the damsite. At the time of construction, this rhyolite was being widely used in the Denver area as a building stone and one block was used in the dam to inscribe the Chief Engineer’s name for the dedication. The rhyolite is easily worked into rectangular blocks with excellent durability characteristics (Figure 19). Creating facing stones in the size required for Castlewood Dam and subsequent transportation to the damsite may have been more expensive; however, the choice of this rock for facing stones would have improved the overall quality and integrity of the dam structure. Further, the uniform surfaces would have allowed for dependable mortar contact and would have reduced the quantity of mortar required between the downstream facing stones.

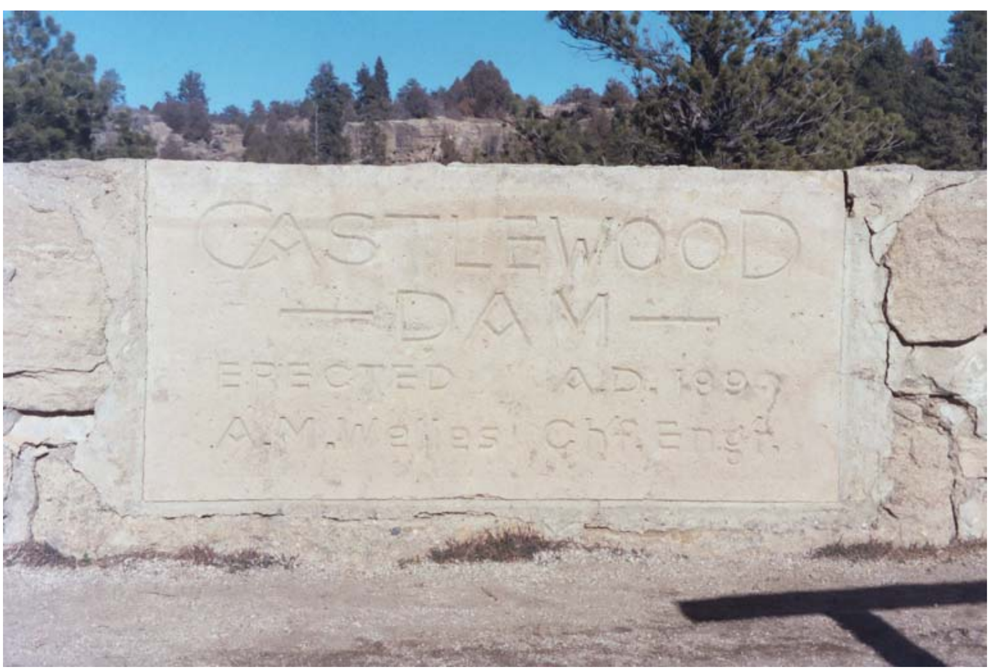

Figure 19. Large rhyolite dedication block. Note contrast between edges of rhyolite and adjoining conglomerate blocks.

Design and Construction practices

Castlewood Dam was not constructed in the accepted manner for rubble-filled, cyclopean masonry dams, such as contemporary Chesman Dam in Colorado and Old Roosevelt Dam in Arizona. Accepted practice used the rubble fill only to take up space to reduce the volume of mortar required; not to completely eliminate the need for mortar. Also, with this method of construction, accepted practice was to embed each of the large rock-blocks in a thick bed of mortar then to place the mortar between the previously set rock-blocks to make a solid, cohesive structure without voids. Castlewood Dam was not constructed in this manner. It appears that Castlewood Dam was constructed such that the upstream rows of facing rock-blocks formed the only water barrier; even though the integrity of the downstream rows of this wall was questionable, at best. The downstream, stair-stepped masonry shell was also constructed with the minimal amount of interstitial mortar and appears only to have been constructed to keep the rubble fill in place and to provide some protection during spillway flows and overtopping. Today there is little remaining interstitial mortar on the exposed downstream slope, attesting to the poor quality of mortar. It is also quite evident that the upstream face of the dam has been patched more than once.

It appears that little foundation shaping or treatment was performed. The right abutment slopes appear to have been shaped, but only to give a uniform foundation upon which to set the first course of conglomerate blocks. On the left abutment, it appears that any smaller conglomerate boulders found in the foundation were used in the masonry or rubble fill. At the same time, the larger residual conglomerate boulders were left in place, probably due to the excessive effort required to remove them, and the dam structure was draped over and around them. Two of these very large boulders are still in place at the outlet works and at the very top of the left abutment, adjacent to the auxiliary spillway and probably prevented the left abutment of the dam from washing out during the first use of this spillway. Seepage would tend to be higher during operation with this irregularly shaped foundation.

Cutoff Trench

The designer/construction engineer states emphatically that a 6 ft (1.8 m) cutoff trench was incorporated into the design and construction (Horan, 1997) and other sources indicate that the upstream wall rests on a concrete footing 5 to 22 ft (1.5 to 6.7 m) below ground level (USGS, 1899). No evidence of this cutoff trench could be found in the portions of the dam that can be observed. At the base of the dam, in the lowest part of the structure, only traces of a leveling-pad can be seen under the upstream masonry wall (Figure 20). Because of the smooth sides of this leveling-pad, it appears that this leveling-pad was not excavated into rock and the smooth bottom surface indicates that the foundation was shaped to lessen the amount of leveling-pad required. The only area of the remaining dam where excavation into rock is apparent and the structure tied into the rock side cut are in the areas of the reconstructed intake tower and intake pipe, the gate chamber, and the outlet-works. In these areas, the rough texture of the outside face of the concrete indicates that the concrete was placed directly against natural materials. It is also curious that the failure of the dam was arrested in this same area. Apparently, the stiffness of this triangular-shaped control structure and the cohesive mass of this thoroughly mortared rock-block structure were sufficient to prevent erosion further into the left side of the dam. However, some of the outlet-works, portions of the downstream masonry stair stepped face and portions of the rubble fill were removed just to the left of the outlet works during and after the failure flood, probably by eddying currents. Another reason for this area withstanding the failure forces was the very large block of rock left in place just to the left of the outlet-works.


Figure 20. Leveling pad at center base of dam. Note mudstone left in place immediately below leveling pad.

Descriptions of the design and construction indicate that the dam was founded anywhere from 6-to-22 ft (1.8 to 6.7 m) below the top of bedrock. A notch averaging about 6-ft deep, below the top of rock, can be observed on the right side where the dam is missing; but this notch covers the entire width of the dam and is either the result of the immediate dam failure, or was made for ease of construction. This notch was not a functional cutoff trench since it is much wider than the base of the dam.

On the downstream left-abutment toe, the lower blocks of rock appear to have been placed directly along the natural contours of the slope with no apparent embedment into the rock. Also on the left abutment, the dam was draped over the existing slope as evidenced by the very large rock-blocks left in place in the outlet-works and in the left abutment spillway areas; giving further doubt of the existence of a cutoff trench in this area.

Early Signs of Distress

We know from photos and contemporary accounts that the dam started to leak excessively, mostly on the left side, upon completion and first filling and that the reservoir was difficult to fill and keep full. We can also surmise; due to the lack of records, that the dam spilled very rarely. This difficulty in filling and very infrequent spilling probably prevented the dam from failing earlier than it did. Although excessive seepage is not encouraging, it was probably contained within the porous rubble fill and apparently did not start removing the soft Dawson Arkose foundation on the left abutment. Conversely, there was no early detection of seepage on the right abutment; but this is the area in which the portion of the upstream, right abutment masonry wall failed due to undermining. Also, this is the area in which the dam ultimately failed, although it took nearly 43 years to transpire.

Six months after completion, the dam developed tiny fissures along the crest that required patching. This type of distress is usually an early sign of settlement. Today those tiny fissures are much larger. However, the design engineer in a letter to the editor stated that "the matter now referred to is simply the separation by a little settlement of the lower or loose rock half of the coping, which was set in 1898 (8years after completion) and has no more to do with the dam's stability than a crack in the city hall. It is four feet above the surface of the reservoir when the dam is overflowing. Likewise in seriousness of character is the 'bulge' mentioned, which is merely an irregularity in carrying up the outer slope" (Horan, 1997). Closer attention to these *minor defects* would have revealed more serious problems in the foundation.

Poor Maintenance

The Denver Land and Water Storage Company, responsible for building and maintaining Castlewood Dam, went bankrupt in 1901, shortly after the required reconstruction of the dam. From that time until 1912 ownership passed through different hands, mostly land speculators from the east. From 1912 till 1930 the dam and appurtenant structures were tied up in court until finally in 1930, the irrigators took over ownership and maintenance of the dam (Horan, 1997). As a consequence of this continual change in ownership, very little maintenance was probably performed on the dam and most likely, no one was inspecting the dam or looking out for serious signs of distress.

Potential Failure Modes

The three most likely failure modes for Castlewood Dam include: 1. A single thickness wall concentrating the seepage at critical points; 2. Internal seepage and foundation erosion; and 3. Erosion at the toe during overtopping.

Potential Failure Mode 1 - Building a large dam with only a thin, single wall acting as the water barrier is a situation that is avoided in current dam design. This thin, single wall tends to shorten the seepage path and concentrates critical seepage flow at a critical point in the foundation; thereby increasing the seepage forces to dangerous levels. Without some type of mitigation this situation can lead to rapid failure of a dam. This situation is especially dangerous with an easily erodable foundation such as the Dawson Arkose without multiple foundation defenses. Usually, this type of failure occurs shortly after completion and during first filling. The fact that Castlewood Dam survived for nearly 43 years before it finally failed diminishes the possibility of this type of failure mode. Further, it appears that the severe leakage observed on the left side of the dam came through the barrier wall rather than through the foundation. However, the early partial failure on the upper right abutment can possibly be attributed to this type of failure.

Potential Failure Mode 2 - A second potential failure mode involves the large amount of seepage seen exiting the downstream face of the dam accompanied by foundation erosion.

Immediately upon completion and initial filling, excessive leakage was observed exiting the downstream face of the dam. This leakage was so extensive that the owners were unable to fill or keep the dam filled for long periods of time. The leakage became so economically worrisome that reconstruction of the dam was initiated in 1898 with the express purpose of eliminating this downstream leakage. Given the very large seepage volumes, the very erodible character of the foundation in the Dawson Arkose, the high porosity and segregation of the rubble fill, a high potential for foundation erosion and failure would seem plausible. Again, this type of failure usually occurs within a short period of first filling, especially given the highly erosive character of the foundation. The cracks in the crest and the bulging at the toe of the dam, observed shortly after completion, are signs of this type of foundation distress. However, the dam probably did not fail in this manner; as, it remained intact for nearly 43 years before it finally failed during an intense thunderstorm event. Interestingly, the right abutment of the dam did not exhibit the large seepage quantities that were observed on the left abutment; although, it partially failed due to undermining early on the life of the structure. It could be, although unlikely, that it took almost 43 years for the seepage in the right abutment to reach a critical stage.

Potential Failure Mode 3 - There has been much debate concerning overtopping of Castlewood Dam, even the chief engineer's comments on this subject seem vague. Since the primary spillway was a 4-ft (1.2 m) deep by 100-ft (30.5m) long notch in the crest at the center of the dam, any flow through this notch could have been interpreted as overtopping of the dam. Also, since the notch was only 4-ft (1.2m) deep with no controls, any flow through the notch could likely rise 4 ft (1.2 m) more and actually overtop the structure. Presently, there does not appear to have been excessive erosion at the downstream toe of the left abutment as would be expected from an overtopping situation. Likewise, on the right abutment, the remnants of the right abutment foundation do not appear to have been eroded extensively during overtopping and failure. The ability of the foundation to slough and heal itself has apparently masked the small amount of erosion that occurred during failure on the right and left abutments of the dam. According to the dam tender, just before the dam failed, there was a *torrent* of water flowing over the top of the dam and within a few minutes, the reservoir level had fallen 13 ft (4 m) (Graham, undated). These are as near to the actual failure conditions that we can get and we can conclude that the dam probably overtopped just before it failed. Also, just before failure, water must have flowed over the right spillway, created during reconstruction of the dam. However, very little evidence of erosion in this spillway is observable today.

Likewise, the last recollections of the dam tender were of "deep rumblings" within the dam just before he left to warn the citizens downstream (Horan, 1997). Hence, the most likely failure mode involved overtopping of the dam and subsequent headward erosion starting at the downstream toe. From the condition of the remaining structure it is most likely that failure occurred somewhere between the center portion of the dam and higher up on the right abutment. Pinning down the actual method and location of failure is more difficult. Either internal erosion due to excessive leakage or disruption of the downstream masonry shell is possible. More likely was rapid erosion of the soft Dawson Arkose at the extreme downstream toe, leading to displacement of the stair steeped rock-blocks in the downstream shell and eroding headward until there was no support for the upstream masonry wall, ultimately leading to toppling of the wall and failure of the structure.

Another scenario within this same mode of failure might have involved the deterioration of the mortar between the downstream masonry over the 43 years of operation with little or no maintenance. In this scenario, the water spilling over the top of the dam, in the spillway area mostly, could have penetrated between the poorly mortared rock-blocks and could have started to pry these blocks apart, removing the rubble fill and ultimately eroding the soft Dawson Arkose. Further headward erosion would have resulted in toppling of the upstream masonry wall, leading to ultimate failure of the dam.

From the reconstruction records, a heavily grouted apron was placed along the downstream toe below and to 50 ft (15 m) either side of the spillway notch in the center of the dam to prevent erosion of the downstream toe (Schuyler, 1909). No evidence of this slab exists today. During previous reported overtoppings, this slab must have been in place, as further reconstruction of this feature is not mentioned in the records. Hence, initial erosion and ultimate failure must have occurred to the right of this apron and progressed rapidly to the right and left of the initial failure location.

If there is a "smoking gun," it has to be in the foundation. The very center of the dam was founded on a mudstone bed rather than on the sandstone beds of the Dawson Arkose. Neither of these two rock types in the dam foundation would be considered as a suitable foundation for any water retention structure without considerable treatment and multiple defensive measures to insure integrity; however, the mudstone was by far the weakest of the two materials and no apparent attempt was made to excavate 10-ft (3 m) deeper to found the dam on the slightly more competent sandstone or to at least excavate a cutoff trench through the weak mudstone. If the dam began to fail anywhere close to the center section or just to the right of the spillway notch, the mudstone would have been a contributing factor.

CONCLUSIONS

In order to adequately judge the design and construction of Castlewood Dam, we need to put the design and construction in the perspective of its time. In the late 1800s, cement and transportation were very expensive and labor was relatively cheap. Also, the consortium of eastern investors and local land owners that built the dam was probably short of funds and likely, made every effort to cut costs. The cumulative affect of these cost cutting efforts is evident in the remains of Castlewood Dam. The use of cement was kept to an absolute minimum, as evidenced by the poor filling between rock-blocks, the extensive use of chinking to reduce the amount of mortar, and the complete lack of mortar anywhere in the rubble fill. In addition, the use of native materials was maximized without sufficient efforts to obtain the best native materials, which were available on the second ridge to the west in the form of the Castle Rock Rhyolite; even though added transportation costs would have been incurred. Instead, irregularly shaped conglomerate blocks were used which resulted in irregular spaces and filling between blocks leading to excessive leakage and poor bonding between blocks and which ultimately lead to foundation erosion.

Weather was probably a factor in the poor construction of Castlewood Dam. The dam was started in the winter of 1889 and completed in 11 months. This excessively rapid construction schedule for such a large structure meant that construction progressed through a Colorado winter with below freezing temperatures and frequent snow falls. In this period of dam building, construction was usually stopped during winter months as insulation methods, such as insulating blankets, had not yet been devised and other insulation methods, other than straw covering, were not available to protect the concrete from freezing. Consequently, much freezing of the interstitial mortar must have occurred; resulting in shrinkage, cracking and ultimately leading to excessive leakage and questionable structural integrity.

The designer/construction engineer did not understand or respect the foundation rock and did not build the dam within the parameters that the foundation required. Also, short cuts were taken during construction, the most serious being the low quantity and quality of mortar and concrete. These short cuts along with misunderstanding of the foundation led to serious problems immediately upon completion, including settlement and movement of the rock-blocks, undermining of the upstream masonry wall and extensive seepage requiring reconstruction. These defects combined with probable poor maintenance, finally led to failure of the dam during a severe thunderstorm and overtopping event. Another problem involved the designer/construction engineer's adamant refusal to admit that his dam may have been defective following repeated indications to the contrary.

The failure of Castlewood Dam also draws attention to the fact that in Colorado, as in much of the west, the most dangerous periods for dam safety occur during the summer months. The intense summer thunderstorm events, not the rain on melting snow events that occur in the spring of the year, produce the dangerous floods that can destroy dams.

The failure of Castlewood Dam is an example of how failure to analyze the foundation, failure to incorporate appropriate and multiple foundation design parameters, combined with questionable design and construction practices and failure to design for appropriate flood events can have catastrophic results. We continue to learn from our mistakes and strive to incorporate these lessons in the design of our major dams to prevent this type of failure that lead to the devastation and destruction in Denver from ever reoccurring in the future.

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INVESTIGATIONS AND DESIGN FOR REPLACEMENT OF THE OUTLET WORKS STANDLEY LAKE DAM AND RESERVOIR JEFFERSON COUNTY, COLORADO

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Key Words: tunnel, portal, shaft, claystone, slickensides, roadheader, TBM, microtunneling, design, construction, slope failures

ABSTRACT

Standley Lake is a large water storage reservoir in the northwestern part of the Denver Metropolitan Area. The reservoir is the main raw water storage reservoir for the Cities of Northglenn, Thornton, and Westminster. The reservoir also stores agricultural irrigation water for the dam and reservoir's owner, the Farmers Reservoir and Irrigation Company (FRICO).

In 1996, comprehensive geotechnical investigations were begun to address some of the deficiencies in the existing dam embankment and appurtenant facilities. The investigations were planned to address embankment stability and inadequate spillway capacity; and to provide information for designing a new tunneled outlet works on the dam abutment, separated from the dam embankment. All the work had to be designed so that the new facilities could be constructed while maintaining a full reservoir and uninterrupted water deliveries to the Cities and FRICO. Consequently, the new outlet works requires wet taps into the reservoir.

Understanding the site stratigraphy, and the physical and engineering properties of the bedrock were important factors in developing the outlet works designs. The tunneled outlet works and valve shaft designs had to take into account the swelling potential of the claystone bedrock, the slaking behavior of weak rock when exposed to air and water, erodability, and the potential for encountering weakly cemented or uncemented sandstone lenses in the underground work.

Designs for the Standley Lake Rehabilitation were completed in the spring of 2002 and approved by the Colorado State Engineer's Office. Construction of the project facilities started in August of 2002 and is expected to take about two years.

This paper summarizes the dam history, the geologic setting, and the engineering geologic investigations and analysis required to prepare the outlet works design.

The paper also provides a summary of the construction completed as of the writing and the geologic conditions encountered.

INTRODUCTION

Standley Lake Dam and Reservoir is located in parts of Sections 16, 17, 20, 21, 22, 27, 28, and 29 in Township 2 South, Range 69 West, Jefferson County, Colorado. This site is in the northwestern part of the metropolitan area of the greater Denver Metropolitan area. The areas, particularly downstream, are developed with residential housing. Figure 1 shows the reservoir and general area surrounding the project.

The dam in its present configuration is an earth embankment with a structural height of 110 ft (33.5 m). The embankment has a crest length of 6,600 ft (2,012 m), and a crest width of about 20 ft (6.1 m). The embankment is constructed of 3.5 million cubic yds (2.9 million cubic m) of embankment fill. The upstream slopes of the dam are relatively steep near the crest, and flatten into the reservoir. Downstream, the upper embankment is approximately 2.5:1 (horizontal to vertical), with a 4:1 toe berm extending two-thirds of the embankment height. In the area of the downstream valve house, there is no toe berm, and the entire slope is 2.5:1 or steeper.

The reservoir stores 42,000 ac-ft (51.8 million cubic m) of raw water. The stored water is mainly offstream storage delivered to the reservoir from Clear Creek via a system of irrigation canals. The reservoir is the major raw water reservoir for the Cities of Thornton, Northglenn, and Westminster; and also stores irrigation water for the dam and reservoir's owner, Farmers Reservoir and Irrigation Company (FRICO). The reservoir is operated jointly by the cities and the owner through the Standley Lake Operating Committee (SLOC).

DAM HISTORY

The dam was originally constructed starting in approximately 1908, and was constructed over approximately a four-year period. The embankment spans the broad Big Dry Creek drainage. This is a small perennial stream that drains the eastern front of the Rocky Mountains. The Front Range foothills lie approximately eight miles to the west of the site. The original construction was completed by dumping soil from railroad trestles constructed upstream and downstream of the dam centerline to form embankment shells. Between the dumped soil shells, a relatively impermeable clay core for the dam was created using puddled clay. The clay was dumped in the trough between the shell zones, and sluiced with water to breakdown the clay lumps. These construction techniques resulted in an embankment with low and variable density since there was really no control over the density or compaction of the soils. This resulted in a relatively weak dam embankment. The foundation of the dam embankment is relatively weak, Tertiary age bedrock.

From the very beginning, the dam was plagued with a series of slope failures, both upstream and downstream. By 1960, the crest of the dam had settled almost 40 ft (12.2 m). In the mid 1960's, the dam was enlarged and reconstructed. The basic construction plan was to construct an almost entirely new dam embankment on the downstream slope of the failed and settled old embankment.

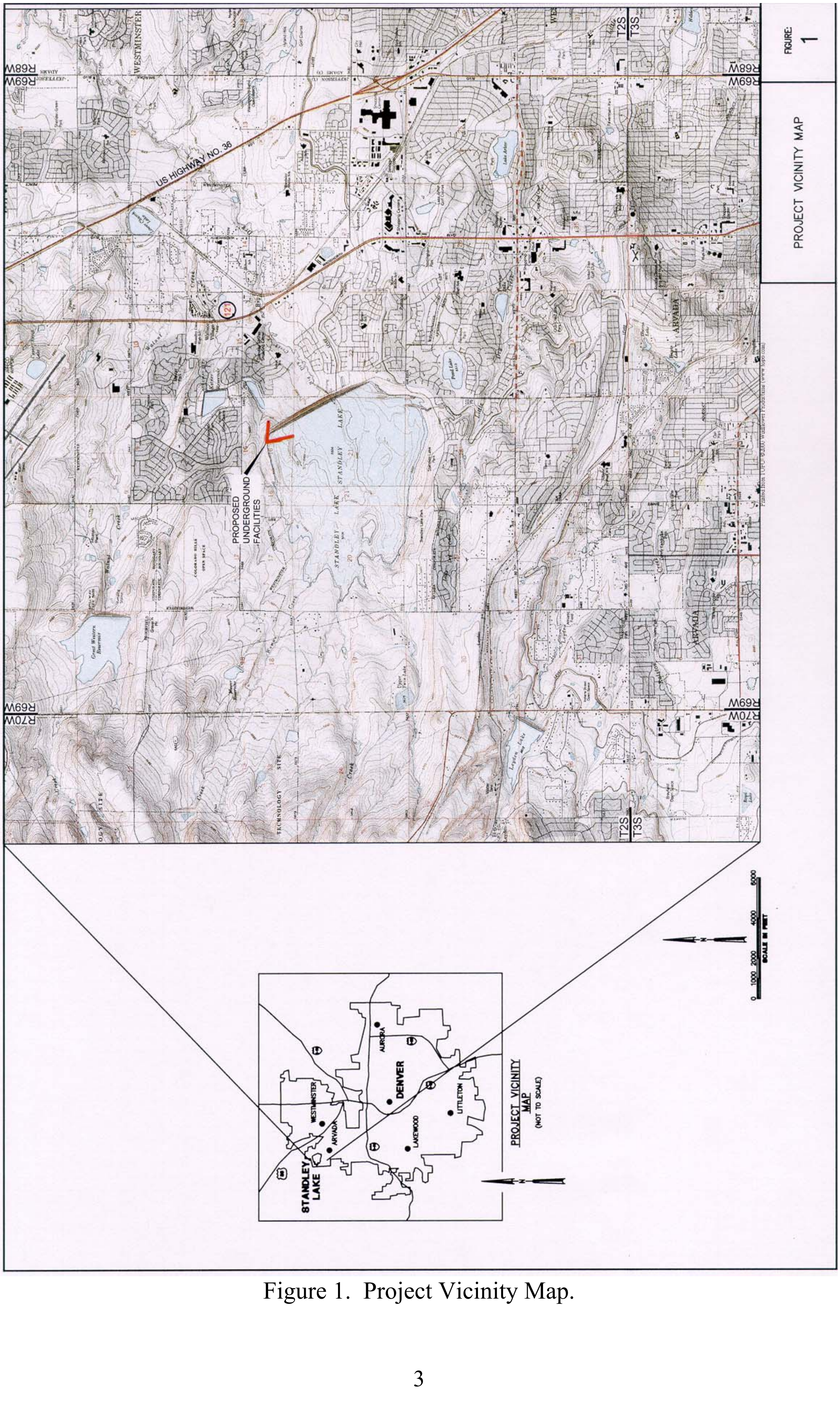

Figure 1. Project Vicinity Map.

The slope failures and settlement problems continued, even with this almost entirely new embankment. The instability problems were remedied to some extent with the construction of an extensive toe berm. However, in the area of the outlet works, the dam embankment remained relatively steep in order to avoid extending the outlet conduits and moving the downstream valve vault further to the east.

The existing outlet works consists of four 48-in (1.22 m) diameter steel outlet pipes with terminate in the downstream valve house. The dam is operated with downstream control, so the conduits are pressurized. In the 1980’s, internal inspections of the outlet conduits showed that separations had occurred in the conduits. The separations were suspected to be the result of the valve house sliding downstream on the weak claystone foundation. Internal flexible sleeves were installed on the insides of the conduits to prevent leakage, and tendons were installed in the valve house foundation to anchor the valve house in-place and resist additional movements. In addition, a more intensive monitoring program using inclinometers, piezometers, surveying, and load cells on the tendons was initiated.

In 1996, comprehensive geotechnical investigations were begun to address the main deficiencies of the dam embankment and appurtenant structures. These investigations were scoped to provide information for construction of improvements. The goal was to renovate the dam and appurtenant structures, enhancing the safety of the dam and reservoir to allow it to function as a reliable storage facility for the next 50 to 100 years. The improvements focused on increasing the embankment stability, spillway improvements to meet the State Engineer’s Office (SEO) requirements, replacing the existing outlet works, and completing an engineered abandonment of the existing outlet works.

The design of the new outlet works had to meet some difficult criteria. Because of the history of embankment stability problems, the new outlet works was designed to be separated from the embankment and dam foundation. The facilities also needed to be constructable without interrupting water service to the cities or draining the reservoir. The new outlet works design incorporated multi-level intakes in order to allow flexibility in the blend of water withdrawn from the reservoir to meet treatment concerns.

CONCEPTUAL DESIGNS

Several tunnel options were considered for the new outlet works. The earliest concepts involved deep tunnels and deep-water cofferdams for the construction in the reservoir. Conventional tunnels under the reservoir, in the relatively weak rock, involved considerable risk; and all the options that involved the cofferdams were very expensive. As the concepts were refined, the options that were considered for further investigation involved:

1. Constructing a valve shaft on one of the dam abutments.
2. Constructing multi-level microtunnel intakes into the reservoir using wet recovery techniques for the microtunnel boring machine, and divers to complete the intake construction.

3. Constructing a downstream outlet tunnel from the valve shaft to a downstream outlet conduit and valve house.

With this general outlet works concept in mind, feasibility geotechnical investigations, including exploratory borings and geologic mapping, were made of both abutments to select the best site. Based upon these investigations, the left abutment (north abutment) was selected. This was mainly because the underground facilities sited on the left abutment would be oriented roughly parallel to the strike of the gently southeast dipping strata. The tunnels would encounter fewer strata on the left abutment.

The conceptual design that was carried forward was to construct a valve shaft on the north abutment of the dam. Two microtunnel intakes were to be constructed at different levels from a valve shaft into the reservoir basin. The intakes were to be constructed using microtunneling techniques and wet recovery, as well as marine construction for the ends of the intakes. In addition, a downstream outlet tunnel was to be constructed from the valve shaft to a downstream outlet conduit and then to a downstream valve house.

The subsurface conditions along the proposed below-ground elements of the outlet works were investigated for the final design by drilling seven marine borings along the microtunnel alignments in the reservoir, and 12 land borings at the shaft and along the outlet tunnel alignment. The marine borings were drilled from a barge-mounted drilling rig, and the borings were cored to elevations well below the proposed intake pipeline elevations. The borings for the shaft and the outlet tunnel were drilled using a truck-mounted drilling rig. In addition to the borings, a test trench was excavated into the side slopes of the existing spillway to expose the bedrock structure and strata in the general outlet works area. A test pit was also excavated near the location of the proposed outlet tunnel portal access shaft near the junction with the downstream outlet conduit. Borings were drilled at the approximate locations on Figure 2. This figure also shows the proposed layout of the new outlet works.

GEOLOGY

General

The Standley Lake site is on the western margin of the Denver structural basin. In the project area, sedimentary strata dip gently eastward away from the mountain front. Bedrock rarely crops out, and is generally blanketed by surficial soils including various alluvial soils, colluvium, loess (or wind deposited) soils.

Surficial Geology

The surficial geology in the project area is varied. In the gently sloping upland areas, the bedrock is blanketed by a variety of Pleistocene age pediment and terrace alluvial soils. These deposits have been mapped in detail on published geologic maps, and differentiation between the units is based on age terrace elevation and general lithology. Pleistocene age alluvial deposits that have been mapped in the project area, include: Verdos Alluvium, Slocum Alluvium, and Louviers Alluvium.

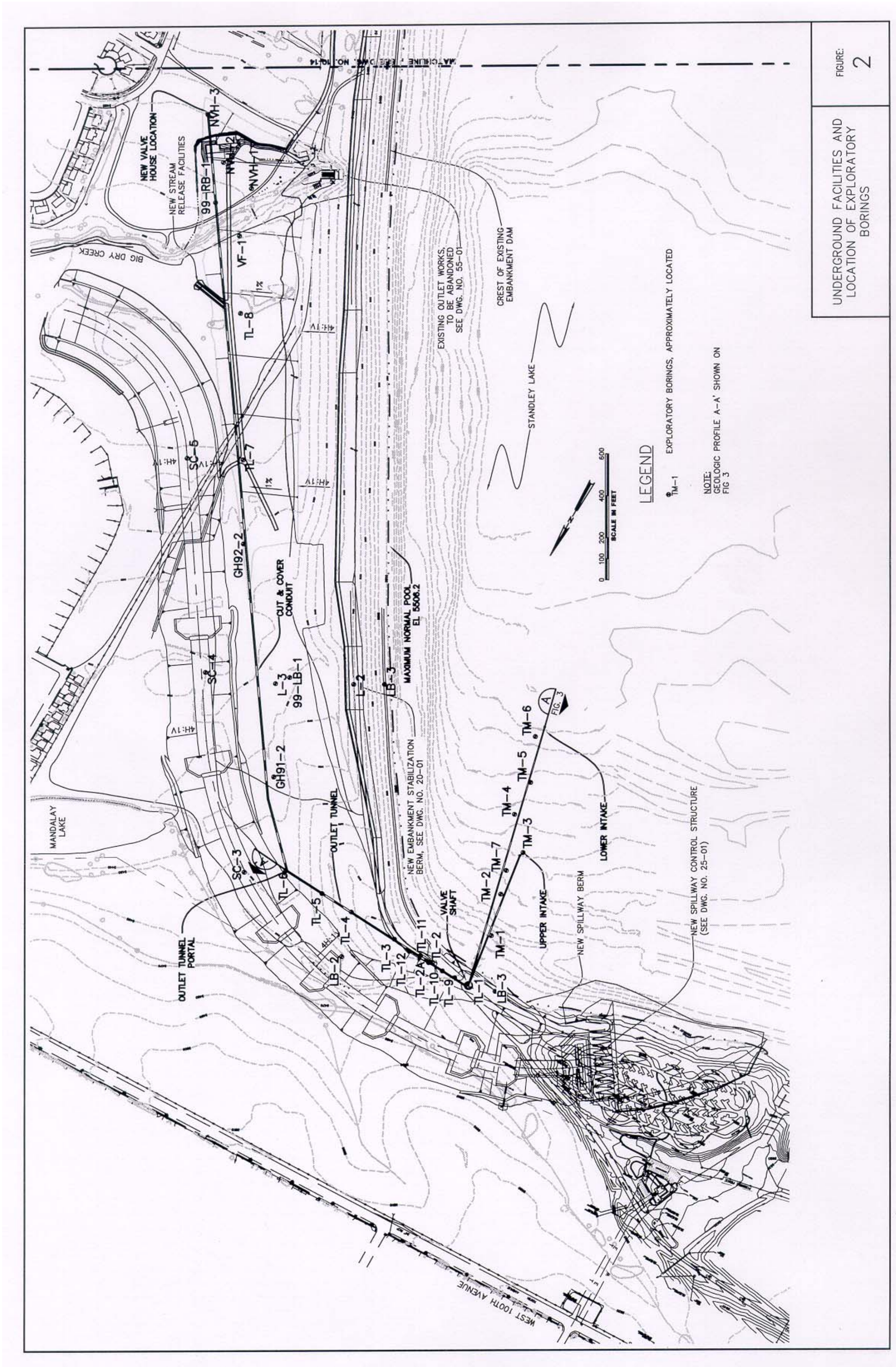

Figure 2. Underground Facilities and Location of Exploratory Borings.

These alluvial soils generally consist of sands and gravels capped with silty clay fine grained soils.

Across the more steeply sections of the site, the bedrock is generally blanketed by a thin layer of colluvial soils. This colluvium ranges from a few inches to a few feet in thickness, and mainly consists of silty, sandy, and gravelly clays.

Loess or wind blown (eolian) soils also blanket the bedrock in the older alluvial soils. These soils generally consist of clays and sandy clays, and are generally less than 10 ft (3 m) in thickness.

Recent alluvium forms the floodplains and low terraces adjacent to the Dry Creek stream channel. These soils consist mainly of poorly sorted sands and gravel overlain by clays and sandy clays.

Bedrock Geology

The near surface bedrock in the project area consists of relatively young sedimentary strata. The bedrock has been mapped as either the Laramie Formation or the Arapahoe Formation. South of the project area, similar materials have been mapped as the Arapahoe Formation. North of the project area, similar deposits have been mapped as the Laramie Formation. On published maps, the bedrock at the site has been mapped as both Arapahoe and Laramie Formation bedrock. Regardless, these bedrock units are non-marine sedimentary beds of the Late Cretaceous/Early Tertiary age. The two formations are similar in lithology, consisting mainly of interbedded sandstones and claystones. The environment of deposition for these sediments was one of coalescing deltas and channel deposits, oxbows, swamps, and shallow lakes. The deposits are characterized by complex interbedding of the sandstones and the claystones with numerous vertical and lateral facies changes. The Laramie Formation has coal seams that were mined in the areas north and east of the site. High angle, normal, and reverse faults are common in the Laramie Formation. The Arapahoe Formation locally contains thin lignite stringers and carbonaceous zones. Both sedimentary formations contain iron nodules in the sandstone facies and iron concretions in the claystone facies. No faults are documented in the project area.

The claystones can be described as highly over-consolidated, stiff, fissured clays. The sandstones are locally very dense, although they are generally uncemented to weakly cemented. Beds of irregular log-shaped features of well-cemented sandstones occur locally. During our investigations of the site conditions, we found that transition lithologies between claystone and sandstone are common. Sandy claystones and clayey sandstones commonly occur throughout the strata.

Interpretation of Geologic Conditions

Geologic information obtained from the exploratory borings was used to develop a geologic profile along the proposed new outlet works. Interpretation of the geologic conditions is illustrated on Figure 3.

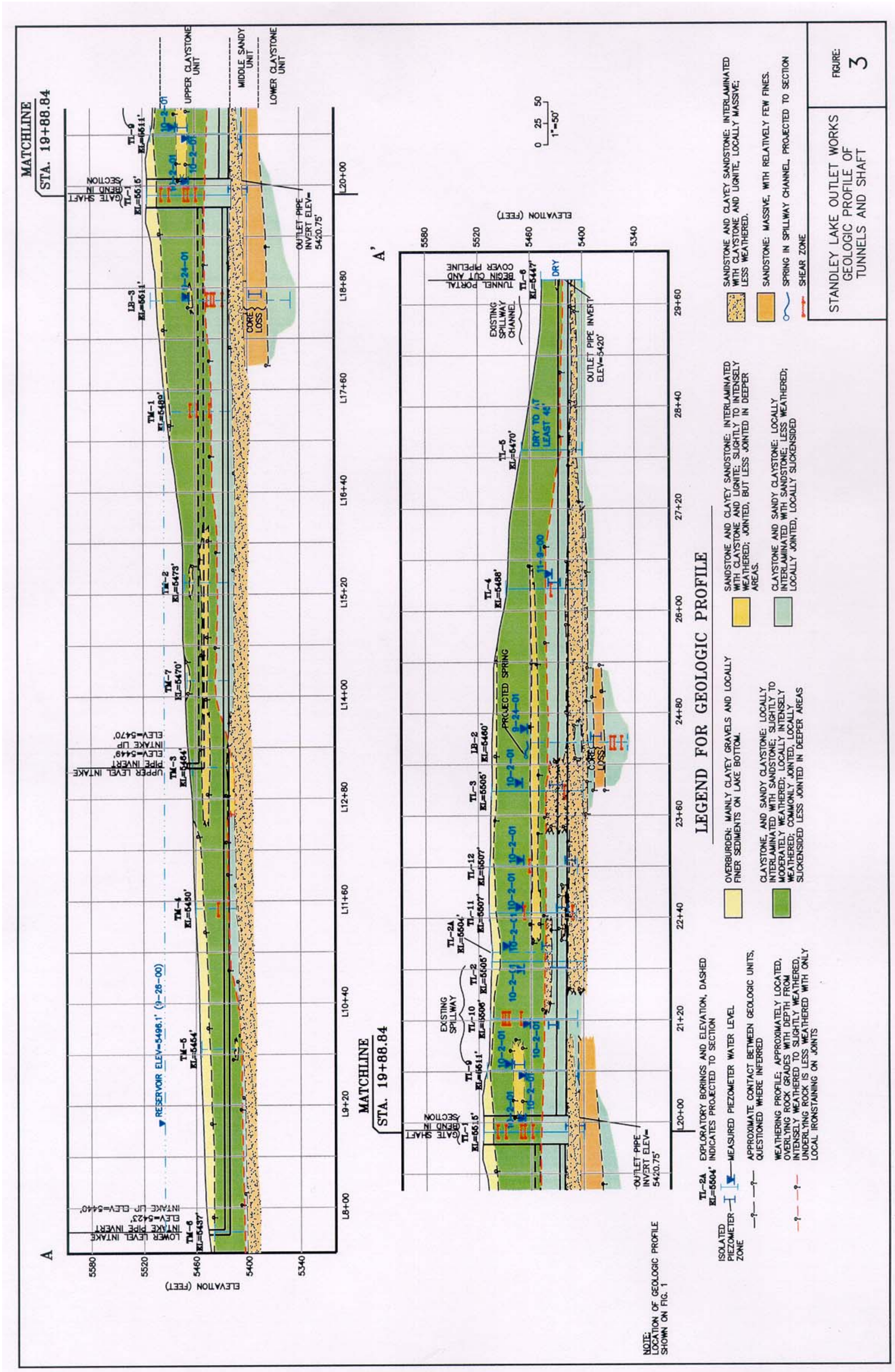

Figure 3. Geologic Profile of Tunnels and Shaft.

Overburden soils are locally absent along sections of the alignment. This is probably due to borrowing of these soils for the construction of the existing dam embankment.

The bedrock generally consists of an upper claystone unit, a middle sandy or sandstone unit, and a lower claystone unit. Most of the borings along the alignment were terminated in the middle sandy unit, while some of the deeper borings penetrated to the lower claystone unit. Our interpretation indicates that the outlet works tunnels and shaft will be constructed almost entirely in the upper claystone unit; although, lenses and beds of sandy claystone, clayey sandstone, and silty sandstone will be encountered. The base of the valve shaft will be near the top of the sandy or middle unit in the clayey sandstone bedrock. None of the facilities extend into the deeper claystone strata.

The borings also indicate that there is well defined weathering profile in the bedrock. The weathering profile is shown on Figure 3. This weathering profile generally follows the topography, but is locally irregular. The weathering of the bedrock is mainly the result of physical processes, including stress relief, differential rebound of the strata, and water infiltration. The bedrock was consolidated under overburden pressures that were well in excess of the existing pressures. The unloading or stress relief of the strata has resulted in differential rebound, the opening of joints, and the development of local fissures and slickensides.

There has also been a breakdown of the bonds (diagenetic bonds) that had developed between the individual particles that make-up the bedrock as a result of geologic time and pressures of deep burial. Water infiltration along the joints and fissures has resulted in iron staining and swelling. Subsequent drying and desiccation has contributed to the additional breakdown of the bonds.

The weathering profile is gradational from the surface of the bedrock where the rock is intensely weathered grading to slightly or less weathered bedrock with depth. At shallow depths, the bedrock has an iron stained fissured soil-like structure, which grades to a very blocky and jointed iron stained structure. Figure 4 shows slickensides and the iron stained joints in the weathered claystone near the outlet tunnel portal. The rock becomes progressively less jointed and less iron stained with increasing depth. An upper weathered zone in the bedrock and a lower less-weathered zone are shown on the geologic profile. The rock in the upper weathered zone ranges from intensely weathered to soil-like to blocky structure, to moderately weathered with describes the less jointed bedrock but with pervasive iron staining, to slightly weathered which described bedrock with occasional iron stained joint surfaced. The rock in the less weathered zone ranges from massive to moderately jointed bedrock with little or no iron staining on the joint surfaces, and little in the way of other visible indications of weathering. The term "less weathered" is utilized rather an "unweathered" because some intact loss of rock strength has occurred over the truly unweathered claystones encountered at greater depths. Another indication of the weathering is the rock in the upper zones tends, in particular the claystones, tend to have higher moisture contents and lower dry densities than the bedrock in the less weathered zones. In the weathered zone, the claystones generally have moisture contents ranging from 14% to 16%, and dry densities in the range of 110 to 120 pounds per cubic foot (pcf) (1.76 to 1.92 g/cm^3). In the less weathered zone, the claystones generally have lower moisture contents and higher densities.

Figure 4. Weathered Slickensided Claystone at the Outlet Tunnel Portal Area.

Groundwater Conditions

The intake tunnels will extend beneath the existing reservoir and so will be below anticipated water levels. Static groundwater levels at the other facilities were encountered well above the base of the structures. At the valve shaft, groundwater was encountered approximately 50 ft above the base of the shaft. Along the outlet tunnel alignment, groundwater levels generally ranged from 30 to 50 ft (9.1 to 15.2 m) above the proposed tunnel invert. Downstream of the valve shaft, the groundwater level drops in elevation mirroring the topography. Groundwater was not encountered in the boring drilled at the proposed outlet tunnel portal at the time of drilling.

General Bedrock Index and Engineering Properties

Samples of the bedrock obtained from the exploratory borings were tested for various physical and engineering properties. The properties of the main rock types are summarized in the following paragraphs:

- Claystones: The claystones are generally quite plastic with plasticity indices ranging from approximately 16% to 45%, and averaging about 32%. The lower plasticity indices were generally measured for the sandy claystones. The claystones slake readily upon exposure to air and water. Tested samples had slake durability indices ranging from about 0.09% to 48.6%, but of the six samples tested, four were less than 1%.

Unconfined compressive strength tests are generally used for index properties on tunneling projects. The claystone bedrock on the site has unconfined compressive strength values ranging from 25 psi (0.18 MPa) to 893 psi (6.33 MPa), averaging slightly more than 200 psi (1.42 MPa). The claystones classify as very low strength rock. Iron concretions occur locally in the claystones. These concretions are generally a few inches in diameter and are generally spheroidal in shape. The concretions are moderately well cemented and are considerably harder than the surrounding claystones. Where observed in test pits in the weathered zone, the concretions were fractured and could be broken with a geologic pick. Gradation tests indicate that up to 99.8% of the claystone is finer than the No. 200 sieve (silt and clay sized material). Hydrometer tests indicate that about 50% of the fine grained materials are clay (-2 micron in size).

Two samples of the claystone were tested for x-ray defraction analysis. The major clay component of the claystone is a smectite rich layer of illite-smectite clay. Lesser clay components include illite and kaolinite. Smectites are a type of clay mineral that is moisture sensitive, expanding when wetted, and shrinking when dried. Smectites include sodium montmorillonite, which is better known as bentonite. This material is commonly used for drilling mud, slurry wall construction, and pond liners because of its swelling properties and its low permeability. In the Denver Metropolitan area, smectites in foundation soils and bedrock are very common. The swelling behavior of the soils has locally caused severe structural damage to commercial and residential structures, as well as roadways.

The claystone bedrock should be relatively incompressible over the range of foundation loads normally anticipated for structures. However, the mixed layer smectite clays are expansive. Several swell-consolidation tests were conducted on the claystones. The claystone has low to moderate swell potential with swell percentages averaging about 2%. However, a few samples had swell potentials of up to 7.5%. Swell pressures were for claystones samples with relatively high plasticity indices, and relatively high initial dry densities and low initial moisture contents. Two remolded samples of the claystone were tested for dispersive properties. Test results indicated that the claystones may be subject to piping, and precautions are necessary around conduits and other structures that might provide seepage paths. In general, the natural moisture content of the claystones is well below the plastic limit of the material, in the range of 11% to 15%. In sheared zones with slickensides, the moisture content is likely to be above the plastic limit.

- <u>Sandstone</u>: The claystones are locally interbedded with sandstones. The sandstones generally range from fine to medium grained and from clayey to silty. The sandstone beds are generally weakly cemented to moderately cemented. Some zones encountered in the borings below the depth of the proposed facilities are friable (can be crumbled with finger pressure) to uncemented, but very dense. These weakly cemented to uncemented beds were difficult to recover in the core borings, often being washed away with the drill fluid. A very hard, dense, sandstone bed was encountered in Boring TM-4 just below the tunnel horizon. This bed was approximately one-foot in thickness. While these very hard, well cemented beds are not common, similar beds up to 2 ft thick were encountered in other areas of the site and in the project area. The sandstones classify as low strength rock where the beds are uncemented to weakly cemented, and a low to moderately strength rock where the rock is well cemented. Unconfined compressive strengths for the sandstones ranged from 178 to 5,166 psi (1.26 to 36.6 MPa); but based on previous experience on this site, samples have been tested with unconfined strengths of up to 8,000 psi (56.7 MPa).

- Transition Lithologies: Clayey sandstone and sandy claystone beds are often present as transition lithologies between claystone and sandstone. These beds are generally weakly to moderately cemented, but are generally stronger than the claystones. Some samples had the visual appearance of sandstone in the field, but when tested for gradation in the laboratory, the classification of the samples were very sandy claystones. It was also common to observe sandstones that displayed a variegated or mottled appearance as they contained small pods and veins of claystone within a sandy matrix. Thin lignite stringers and carbonaceous zones also occur locally in the bedrock strata.

GENERAL DESIGN OF OUTLET WORKS ELEMENTS

Since claystones will be the majority of the bedrock encountered in the construction of the outlet works, certain properties of the bedrock strongly influence the designs. These properties include:

- Swell potential
- Slickensided and jointed rock in the weathered zone
- Strength
- Slaking behavior
- Dispersivity
- Corrosivity

Also, the potential for encountering gases, mainly methane and sulfur dioxide, needed to be considered.

Valve Shaft

The valve shaft is 40 ft (12.2 m) in excavated diameter with an excavated depth of approximately 105 ft (33 m). Because of the location adjacent to an existing dam, blasting was not considered for excavation. Mechanical excavation was required. Pre-construction dewatering was planned at the shaft to draw the water level below the invert of the structure. The designed primary support for the shaft consists of a combination of steel fiber reinforced shotcrete, W8 by 40 pound circular steel ribs on 4-ft (1.2 m) centers, and grouted rock dowels. The initial shotcrete is specified to be applied within two hours of excavation to prevent slaking. The grouted rock dowels will be drilled through the shotcrete to prevent loosening in the rock surrounding the shaft. The steel ribs will be installed inside the shotcrete, and the ribs will be encapsulated inside a second layer of shotcrete. The primary support is designed to support all of the anticipated ground loading expected during construction.

After the shaft has been excavated to final grade, a reinforced concrete lining, 27 in. (0.69 m) thick will be placed on the interior to the primary support and will incorporate the primary support into the final structure. The combination of the final concrete lining with the primary support has been designed to support all anticipated lateral loads, including swell pressures. In addition, the reinforcement of the concrete has been designed to provide a jacking pad opposite the microtunnels to resist the jacking pressures that will be required (up to approximately 1,200 tons (1,090 metric tons)) for microtunneling the intake tunnels.

In order to reduce the possibility of seepage around the valve shaft, a grouting program has been designed to fill any voids that may have developed from excavation and loosening of the bedrock around the shaft.

Intake Tunnels

The intake tunnels were designed to be constructed using microtunneling techniques. The final tunnels will be lined with steel conduits 72 in. (1.82 m) in diameter. With microtunneling, the tunnel is advanced into the ground using a microtunnel boring machine (MTBM) with a rotating cutting head. Thrust is transferred to the cutting head through the steel pipe advanced behind the MTBM. The thrust is applied to the pipe using hydraulic jacks pushing-off the shaft wall. Cuttings are removed from the MTBM using a pipeline and a slurry thickened with bentonite and polymers.

The steel intake conduits at Standley Lake that will be used to advance the MTBM have a 3/4 in. (1.9 cm) wall thickness. This relatively thick pipe was selected to allow high jacking pressures because of the swelling properties of the claystone. This pipe will provide the anticipated jacking load requirements plus a factor of safety. Corrosion resistant coatings have been specified. In addition, the steel intake pipes will be cathodically protected.

Upper Intake Tunnel

The upper intake microtunnel is approximately 629 ft (192 m) in length. The general sequence will be to bore-out from the valve shaft with the MTBM followed by the steel pipe. Cuttings removed from the cutting head will be pumped to the surface where the slurry will be cleaned of cuttings (solids) using settling basins and other methods, and returned back down the shaft to the tunnel boring machine as the drilling progresses. As the boring advances, pipe will be added in 20-ft (6.1 m) long segments. When the machine approaches the underwater end of the intake pipe, the utility lines connecting the machine to the shaft will be disconnected, a bulkhead behind the boring machine will be installed in the pipe, and the machine will be pushed-out into a pre-excavated trough on the bottom of the reservoir with what is know as a “blind shove.” Divers will then go down to the exposed MTBM, disconnect the machine, and hoist the machine to the surface. The remainder of the intakes will then be assembled on the bottom of the reservoir using divers and cranes from a barge above. After the intake pipe is in its final position, grout ports in the pipe will be used to displace slurry in the annulus between the pipe and the claystone with cement grout.

Lower Intake Tunnel

The process for constructing the lower intake tunnel will the same as the upper tunnel. The intake pipe is also a 72-in. (1.8 m) diameter steel pipe with a 3/4-in. (1.9 cm) wall thickness. The length of the intake is approximately 1,229 ft (374.5 m).

The steel pipe provides both the initial and final support for the microtunneled intakes.

Outlet Tunnel

The primary support requirements for the outlet tunnel were analyzed considering the rock conditions, the diameter of the tunnel, the depth of the tunnel, and the potential for swelling properties of the claystone. Machine excavation was specified.

Two types of primary support were designed for the anticipated ground conditions. In Type 1 or generally firm ground, support alternatives consist of W6 x 25 pound steel ribs on 5-ft (1.52 m) centers with hardwood lagging for a full face TBM excavated tunnel, or 4 in. (10.1 cm) of shotcrete for a roadheader excavation. The Type 1 support is planned for the majority of the tunnel. Type 2 support is planned for areas where poorer ground conditions are expected near the tunnel portal and areas of potentially higher stresses near the intersection of the tunnel with the valve shaft. Type 2 support consists of W6 x 25 pound ribs on 4 ft (1.2 m) centers for TBM excavation, or lattice girders on 4-ft (1.2 m) centers encased in 6 in. (15.2 cm) of shotcrete for the roadheader excavation. In addition to the primary support requirements outlined above, pre-support rock dowels will be installed at the downstream portal and at the intersection of the outlet tunnel at the valve shaft to support the intersection. To prevent slaking of the claystone during construction of the outlet tunnel, shotcrete for roadheader excavation or a polymer for TBM excavation will be pneumatically applied to the rock shortly after exposure. Invert protection is specified to protect the invert from deterioration during tunnel excavation.

The final lining for the outlet tunnel will be a welded steel conduit 102 in. (2.59 m) in internal diameter with a wall thickness of 1/2-in. (1.27 cm). External stiffeners will add additional hoop strength to the steel liner pipe. The annular space between the primary support and final support will be filled with low density cellular concrete. Contact grouting will be specified between the cellular concrete and the outlet pipe, and the cellular concrete and the initial support to mitigate the dispersive property of the claystone and prevent seepage from eroding the rock. Corrosion potential will be addressed through isolating the pipe from the claystones through the use of the cellular concrete.

CONSTRUCTION UPDATE

Construction began on the Standley Lake Project in late August of 2002. The construction of the shaft began with the installation of dewatering wells in early September. Excavation of the shaft actually began in late September.

The contractor excavated the shaft using a Caterpillar 312B excavator. The rock excavated easily through the non-weathered claystones to a depth of about 50 ft (15.2 m). From 55 ft (16.8 m) on down, a second excavator with a hoe ram was required to break-up the rock. The rock was then excavated and loaded into a skip pan for hoisting by crane to the ground surface.

The primary support was installed as the excavation proceeded in 4-ft vertical increments. The initial shotcrete layer was applied to the claystone shortly after final trimming of the 4-ft intervals. The radial rock dowels were then drilled and installed, and completed with bearing plates. The steel ribs were then placed using tie rods and collar bracing, and blocked as required. Following the next excavation interval, the full 3 in. (7.62 cm) of shotcrete was applied and the rib was encased. Figure 5 shows the shaft during excavation.

Figure 5. Excavators in Shaft During Construction

Shaft excavation reached its final depth in the third week of January 2003. Most of the materials excavated were claystones, although lenses and layers of sandstone and some hard iron concretions were excavated as the work proceeded. The largest iron concretion was approximately 6 ft by 3 ft near the edge of the shaft. Jack hammers were used to excavate the concretion.

The dewatering wells were effective at drawing down the water in the area of the shaft. The excavation was essentially dry with the exception of two or so minor moist areas, and the seepage evaporated as fast as it could enter the excavation. The weathering horizon extended to a depth of 55 ft (16.8 m). The rock in this upper zone was very prone to slaking. The rapid application of shotcrete to the exposed bedrock prevented significant slaking of the claystone.

In general, the claystone in the weathered zone was firm to slow raveling ground. Some minor sloughing and overbreak occurred on slickensided joints, but this too was limited by the application of shotcrete. Below 55 ft (16.8 m) the bedrock was essentially firm ground and there was very little overbreak.

The contractor is now in the process of constructing the final reinforced concrete lining.

The construction of the outlet tunnel started the week of January 27, 2003. The contractor chose to excavate the tunnel using a roadheader type machine for the excavation and shotcrete and lattice girders for his primary support. The downstream portal for the outlet tunnel was prepared during the weeks prior to January 27th. The machine turned under on January 29th. By January 30th, the tunnel had advanced approximately 26 ft (7.9 m) from the portal. Figure 6 shows the tunnel portal and the road header starting the excavation.

Figure 6. Outlet Tunnel Portal and Roadheader Excavating Tunnel.

The claystone near the portal behaved as firm to slow raveling ground. Shotcrete was applied soon after the rock was exposed to reduce slaking and raveling. The lattice girders were installed on 4-ft (1.21 m) centers and encased in shotcrete as the heading advanced. Figure 7 shows an area where minor fallout occurred from the tunnel crown on a slickensided joint after the initial shotcrete was applied. The roadheader has had no difficulty excavating the claystone. No significant harder concretions or nodules have been encountered thus far.

Figure 7. Minor Crown Fallout on Slickensided Joint.

SUMMARY

The replacement of the Standley Lake outlet works involves significant underground construction of a large shaft, two 72-in. (1.82 m) microtunnels, and a conventional 12-ft (3.65 m) diameter tunnel in an area where recent experience is relatively sparse. The project involves the construction of a 40-ft (12.2 m) diameter shaft approximately 105 ft (32 m) deep, an outlet tunnel approximately 1,000 ft (3.05 m) long and 12 ft (3.65 m) in diameter, and microtunneled intakes beneath a live reservoir. Microtunneling of the intakes will require the wet recovery of the microtunnel boring machines. This project has been termed “wet tap” into an existing reservoir.

The geologic conditions involve underground construction in relatively weak Tertiary age claystone and sandstone bedrock. The design for the underground facilities, both for primary and final support, takes into account the swell potential of the bedrock, slickensides and jointed rock conditions especially in the weathered zone, relatively low strength of the bedrock, the rapid slaking behavior, dispersivity, and corrosivity of the bedrock. The facilities all require mechanical excavation and utilize typical ground support systems, including shotcrete, rock bolts, lattice girders encased in shotcrete, and combinations thereof.

Construction of the outlet works started in September 2002. The excavation of the valve shaft for the project has been completed as of the end of January 2003. The outlet tunnel is presently under construction. The microtunneled intakes are planned to begin construction in April 2003.

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CHANGES IN ENGINEERING GEOLOGY PRACTICES – COMPARISON OF LEMON AND RIDGWAY DAMS, IN SOUTHWEST COLORADO

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Key Terms: geologic practices, earthfill dam, Southwest Colorado, Lemon Dam, Ridgway Dam

ABSTRACT

This paper compares the differences in early exploration of the dam sites, the geologic exploration in the design and construction phases, dam instrumentation, and geologic staffing of two earthfill dams completed twenty-four years apart by the U.S. Bureau of Reclamation (Reclamation). The major changes in engineering geology practices occurred in the technical and field support by geologists during construction of the projects. Many of the changes occurred after studying the Teton Dam failure and incorporating recommendations into construction activities.

Where Lemon Dam had minimal geological construction support and post construction instrumentation, Ridgway Dam had extensive subsurface explorations, instrumentation, and a staff of geologists to document foundation conditions and address geologic problems encountered during construction. A comprehensive instrumentation system was also included in the construction and post construction phase of the project and was designed and monitored by a geologic staff.

Both dam sites had geologic investigations which began with surface mapping, drill holes, test pits, and observations published in early Reclamation reports. During the time period between the construction of these two dams, an increased emphasis had been placed on engineering geology, which greatly affected the incorporation of site geology with dam design and construction practices.

INTRODUCTION

Engineering geologic practices within the Bureau of Reclamation changed over a relatively short period of time (1960's to 1980's). This paper compares the differences in geologic data between two earthfill dams constructed by the Reclamation in Southwest Colorado.

Both dams were authorized under the Colorado River Storage Project Act of April 11, 1956. Lemon Dam is the principal feature of the Florida Project and is located on the Florida River, approximately 17 mi (27 km) northeast of Durango, Colorado, in La Plata County.

Ridgway Dam is the principal feature of the Dallas Creek Project and is located approximately 6 mi (10 km) north of Ridgway, Colorado, on the Uncompahgre River.

DESCRIPTION OF DAMS

Lemon Dam

Construction of the dam involved an earthfill dam embankment, spillway, outlet works, and relocation of county road 284.

Lemon Dam is a zoned earthfill structure with a maximum height of 215 ft (65.5 m) above the streambed and a crest length of 1,360 ft (414.5 m). The dam embankment has a maximum base width of 1,170 ft (357 m), a crest width of 30 ft (9 m), and contains a volume of 3,019,383 cubic yds (2,308,620 cubic m) of earth and rock materials. A cutoff trench to bedrock was excavated for the full length of the foundation with a grout cap and curtain constructed in the bottom of the trench between elevations 7920 ft and 8161 ft (2414 m to 2487.5 m) (Final Construction Report, 1964). Features at the dam include a spillway and outlet works.

The reconnaissance report described dam foundation materials as "red beds" of Triassic and Permian age composed of limestone, shale, siltstone, and sandstone. No detailed descriptions of rocks, joint surveys, or foundation material descriptions were found in any reports.

Construction occurred from 1961 to 1963 and the reservoir was filled in 1965.

Ridgway Dam

Ridgway Dam is a zoned earthfill structure with a structural height of 234 ft (71 m), 2,460 ft (750 m) long, a crest width of 30 ft (9 m) and a base width of approximately 2,500 ft (762 m). This embankment consists of 10,900,000 cubic yds (8,334,140 cubic m) of material, comprised of three general zones. Features at the dam include a spillway and two outlet works structures, all located on the right abutment of the dam. Also included in the dam foundation work was an outlet works excavation and cutoff trench with associated grout curtain and blanket grouting program.

Ridgway Dam is founded in the Jurassic Morrison Formation (Salt Wash and Brushy Basin Members) and Cretaceous Dakota sandstone. Foundation materials generally consist of interbedded siltstone and mudstone. There are sandstone units located in the upper reaches of the dam area.

Construction began in 1978 and was completed in 1987. The reservoir was first filled in 1990.

PRELIMINARY EXPLORATION

Lemon Dam

A brief (4 pages) reconnaissance geological report was published in December 1958 by Upper Colorado Regional Geologists. No feasibility geologic report was found and Lemon Dam was investigated before geologic design data reports were written.

The original axis considered for Lemon Dam in studies first made in 1938-39 was approximately 1,400 ft (427 m) upstream from the present dam location. At that time, exploration was made by two drill holes and four test pits which disclosed that glacial action had deposited beds of sand, gravel, cobbles, and boulders that were extremely permeable. Maximum depth of these deposits was 228 ft (69.5 m). Four additional drill holes were completed on this axis in 1957. The dam axis was moved to the present location after concluding that problems connected to the deep overburden made this axis unsuitable (Reconnaissance Geological Report of Lemon Damsite, 1958).

At the present location, reconnaissance exploration found bedrock to be fairly shallow. Bedrock was exposed on both abutments to varying heights. An outwash terrace consisting of sand, gravel, and large granite boulders existed in the valley floor. Four diamond core drill holes were completed from 1957 to 1958 to investigate foundation conditions. Drilling operations in the valley floor indicated a channel or narrow inner gorge existed approximately 60 ft (18.3 m) below river level and trended across the valley in a southeasterly direction at about 45 degrees with the axis of the dam. Most of the rock at the dam site was shale and siltstone but two beds of limestone were found in the abutments. The quality of the shale and siltstone was suitable as foundation for the dam. The rock was bonded by calcareous cement and was massive, moderately hard, and relatively free from joints and fractures (Figure1).

Some concern was expressed regarding the limestone which showed evidence of solutioning and was thought to contain possible caverns. The possibility of the formation of caves or caverns in the limestone was addressed by performing down-hole percolation tests. The tests indicated both tight rock and areas that were susceptible to losses. In the limestone where solution activity occurred, it was possible to build hole pressures up to 45 pounds (99.2 kg). Other losses were attributed to vertical expansion joints. No faulting or shearing was found in the drill holes (Reconnaissance Geological Report of Lemon Damsite, 1958).

During the fall of 1960, approximately 100 test pits were excavated in the reservoir area from the dam site upstream to Miller Creek, a distance of approximately 1-1/2 mi (2.4 km). These test pits and the previous drill holes were used to determine the type and amount of material available for constructing the dam embankment.

Ridgway Dam

Geologic investigations of the area were begun in early 1949 and identified 3 potential dam site locations. The investigations at the preferred dam site location consisted of surface geologic

Figure 1. Lemon Dam - Photograph shows inner gorge and grout cap.

mapping and subsurface investigations consisting of 8 core holes with associated hydrologic testing. A Reconnaissance Geological Report was published in 1950.

Geologists were closely involved in initial site evaluation and feasibility studies which included site investigation (surface mapping and drilling), test pits and groundwater evaluation as well as geologic feasibility reports (published in 1979). Geologists also prepared reports and evaluations associated with design data reports prior to award of Stage 1. This included surface mapping of the site, drilling, hydrologic testing, landslide analysis, seismic evaluation, and borrow area investigations, to name a few.

Prior to construction of the dam and appurtenant structures, a total of more than 20 exploratory drill holes had been completed. A Geologic Design Data Report was completed in 1981 which summarized the additional geologic investigations conducted prior to the start of dam construction. It was also decided that construction of the dam would be initiated in 2 stages. Stage 1 would generally involve excavation of the channel area, construction of the outlet works, and the placement of embankment to original ground surface elevation. This would also allow design assumptions to be evaluated and reviewed prior to actual construction of the dam embankment. Stage 2 of construction would involve excavation of the abutments and actual dam embankment placement.

A Draft Construction Geology Report (1992) was completed at the conclusion of construction. It contains detailed summaries of geologic investigations completed during Stage 1 and Stage 2 as well as detailed documentation of foundation conditions at the site. Due to budget constraints the draft report was never finalized.

CONSTRUCTION GEOLOGY

Lemon Dam

No geologists were assigned to the construction staff, although geologists from the Upper Colorado Regional Office in Salt Lake City visited the site as requested by the onsite construction engineer for a landslide problem on the right abutment. It was not a practice to write final geologic construction reports, however a final construction report was written.

Lemon Dam was constructed on bedrock of the Cutler Formation (Lower Permian) consisting of red to gray calcareous siltstone, shale, and sandstone with interbedded limestone (Figure 2).

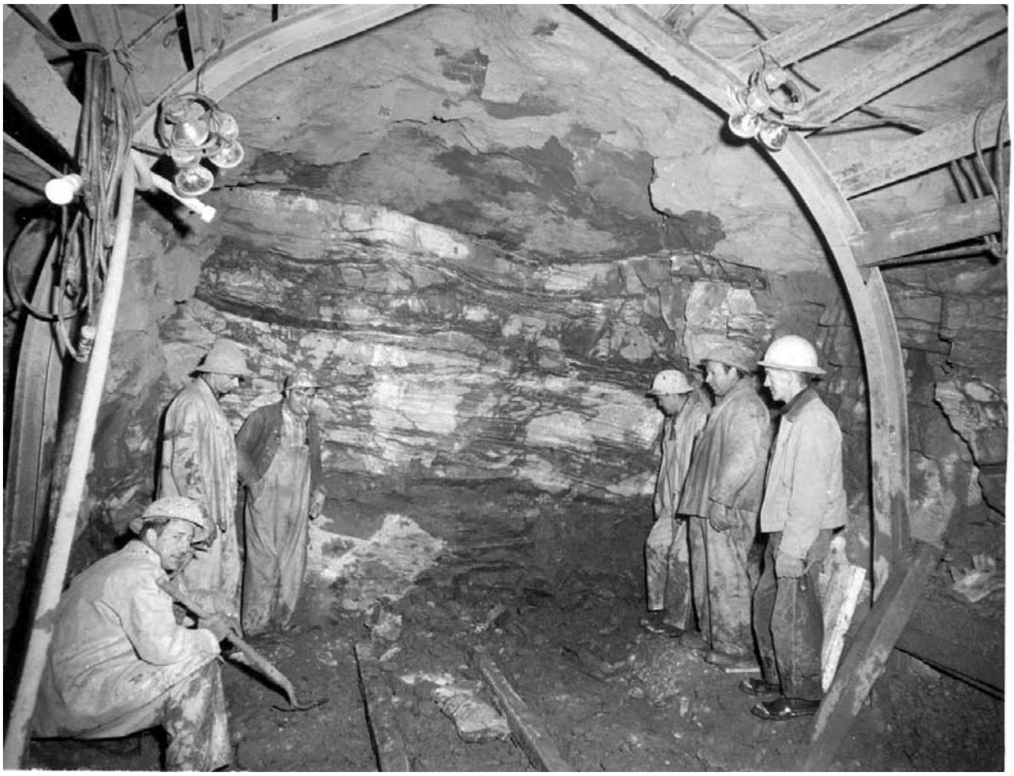

Figure 2. Lemon Dam - Red siltstone of the Cutler Formation exposed in the outlet works tunnel.

The reconnaissance report apparently misidentified the formation or more detailed geologic mapping identified the bedrock as the Cutler Formation. The bedrock is highly fractured and fissile and the shale interbeds readily slake when exposed to air. The dip of the bedrock beneath the dam is approximately 10 degrees to the south. Preconstruction drill hole data indicates that some of the limestone interbeds contain solution channels. Foundation treatment consisted of a grout curtain, with a minimum depth of 160 ft 48.8 m) across the valley floor and up both abutments, and a cutoff trench within the limits of the Zone 1 core and extending 5 ft (1.5 m) into bedrock or a minimum base depth of 30 ft (9 m) (Figure 3). The trench was excavated to a 10-ft (3 m) width in siltstone below the old river channel. Photographs of the construction area can be used to gather missing geologic data. Photographs such as Figures 1and 2 can be used to gather general information on rock units, jointing, rock hardness, bedding, and other attributes. However none of this data was documented during construction.

Embankment measurement points and deflection points are currently the only form of instrumentation at Lemon Dam.

Figure 3. Lemon Dam - A rock saw excavating the grout cap and cutoff trench.

Ridgway Dam

Construction at Ridgway Dam was completed in 2 separate Stages, using 2 separate contractors. This staging process allowed an initial design data package to be completed consisting of initial geologic data recommendations and expected construction conditions. The Stage 1 contract (1979-1982) included general excavation of the dam core area and embankment placement to ground level and outlet works alignment and construction. Detailed geologic mapping of the outlet works excavation and exposed foundation areas were completed during Stage 1. At the completion of Stage 1, design assumptions made during the design data gathering phase were evaluated and modified to fit existing site conditions prior to award of the Stage 2 contract. Due to having the additional preconstruction exploration program, the basic geologic and hydrologic design assumptions were correct. Projected faults in the right abutment area were in close proximity to their locations exposed during construction. Stage 2 (1982-1987) included cleaning of the abutments, tunnel excavation and reinforcement, spillway excavation and actual placement of the embankment material and concrete work (Figures 4 and 5). Initial filling of the reservoir began in 1987 and was completed in 1990.

Figure 4. Ridgway Dam - Foundation clean-up on the left abutment.

Figure 5. Ridgway Dam - Foundation excavation and final cleanup on the right abutment, near dam centerline.

During the construction process, a minimum of 2 geologists (1 resident geologist during Stage 1, 2 to 3 during Stage 2) were present on-site to monitor, direct and inventory construction activities. The geologists were on site on a day-to-day basis and directed final cleanup, tunnel excavation support, and approved final cleanup prior to concrete and embankment placement.

During much of the construction from 1982 to 1987, 3 geologists were located at the Project. The geology staff was also instrumental in preparation of dewatering and unwatering plans, curtain and blanket grouting design and implementation, design and implementation, slope stabilization during the construction process, embankment and abutment instrumentation planning and installation, tunnel support criteria, approval of contractor blasting plans and location evaluation of borrow/quarry sites. All of the foundation, which was excavated to bedrock for Zone 1 material, was mapped using a plane table at a scale of 1in = 20 ft (2.54 cm = 0.78 m). During the initial filling of the reservoir, which lasted 3 years, 1 to 2 geologists were employed at the site. A Draft Construction Geology Report was also prepared at the end of construction to document all mapping and items of concern during the construction process.

During the construction phase, site geologists aided in achieving the best construction methods possible. In several cases, bedded material on the left abutment had moved in mass and appeared as bedrock to the untrained eye. Close examination of this material by geologists showed the "bedrock" material had moved as a block and was unsuitable as foundation material. During excavation of these rock masses, slickensides were noted between the landslide mass and competent bedrock.

During grouting operations, a section high in the left abutment was uplifted by excessive grouting pressures. On-site geologists were able to immediately assess the damage, set up an investigation program, and supply a remediation solution.

SUMMARY

The changes in engineering geology practices (especially geologic monitoring during construction) that occurred between building these two dams were quite drastic. Not only did engineering geology evolve as a science, but knowledge of how foundation materials behaved, the important of rock discontinuities, the role of ground water in the foundation and embankment materials led to a change in the collection of geologic data which was incorporated into the design process. Many of the changes in construction practices and the associated increased involvement of geologists at Reclamation damsites were a direct result of safety of dam considerations which were established after the failure of Teton Dam (1975).

Table 1 indicates that the preliminary investigations conducted early in the stages of the projects were similar. Basic geologic investigations including drill holes to evaluate foundation conditions, location of nearby borrow sources, and identification of potential problems that might cause the dam site to be moved. As the table shows during construction, Ridgway Dam had much more geologic information collected as excavation progressed. A geologic map of the foundation was produced during actual excavation; geologists were on site to assist with unwatering and dewatering, slope stability, curtain and blanket grouting, blasting, and instrumentation installation.

Table 1. Geologic Summary

Investigations	Lemon Dam	Ridgway Dam
Feasibility Geologic Report	Yes	Yes
Feasibility geologic investigations	Yes	Yes
Feasibility drill holes	10	8 (present site)
Feasibility test pits or auger holes	7	16
Geologic Design Data Report	None required	Yes
Design data exploration	No	Yes (20+ drill holes)
Construction geologic mapping	No	Yes
Construction geologists assigned	None	2-3 geologists
Instrumentation	Yes	Yes
Final Geologic Construction Report	None	Yes

At Lemon Dam, very little geologic information was collected during design and construction. Where no geologists were stationed at Lemon Dam during the construction, three were stationed at Ridgway Dam resulting in a significant difference in the amount of geologic data collected and known about the site. Very little data exists on geology, discontinuities, and rock units. Although Lemon Dam currently operates normally, if something unexpected should occur, a geologic exploration program would have to be developed and executed prior to remediation work. This could include exploratory drilling in the abutments and dam to collect information on rock units, discontinuities, faulting, foundation and dam conditions, and piezometric water levels. Surface geologic mapping and subsurface exploration around the abutments and reservoir may be required to collect data on discontinuities, solutioning, or rock characteristics.

Some information could be deduced from old photographs taken of the site and during construction. Photographs taken during construction show general bedding and jointing conditions, seep locations and quantities, and construction practices. In Figure 6, zone 1 material is being placed in the inner gorge area of Lemon Dam. From the photograph, small seeps can be seen in the bedrock on the right side, a sump can be seen in the left center of the photograph. Water entering the excavation appears to be a relatively small quantity. In the bedrock on the right side, several joints can be seen trending sub parallel to the dam axis. Bedding thickness can be estimated based on equipment heights and the cutoff trench width.

Instrumentation is summarized in the Table 2. Based on existing documentation and knowledge of Lemon Dam site, no instrumentation was planned except for embankment measurement points to monitor gross movement of the fill. This would be very unusual for most dams constructed

Figure 6. Lemon Dam - Zone 1 placement in the inner gorge area.

after the mid-1970's, most of which had the means to monitor piezometric levels in the embankment and abutments.

Table 2. Instrumentation Summary

Instrumentation	Lemon Dam	Ridgway Dam
Embankment Measuring Points	9 Embankment	37 Embankment 48 Left Abutment
Base Plates	None	3
Extensometers	None	14
Observation Wells	None	5
Inclinometers	None	14
Total-pressure Cells	None	13
Pneumatic Piezometers	None	73
Vibrating-wire Piezometers	None	6
Porous-tube Piezometers	None	10
Slotted-pipe Piezometers	None	52

Ridgway Dam was constructed with the importance of onsite geologists and as-built information which would ultimately be used for safety of dam analysis. With an onsite geology staff, all aspects of the construction geology could be documented and a wide based knowledgeable staff was utilized to solve construction problems.

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Jim Gates – Project Geologist - Bureau of Reclamation

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STRONTIA SPRINGS DAM
MODERN WONDER

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Key Terms: arch dam, Denver Water, foundation treatment, shear zone

SUMMARY

The Strontia Springs Dam and reservoir, a principal feature of the Foothills Project, is located on the South Platte River, 25 miles southwest of Denver, Colorado. It is owned and operated by the Denver Board of Water Commissioners/Denver Water. The dam was constructed for storage and diversion of water in the most recent expansion project (Foothills) of the water supply system for the Denver Metropolitan Area. The Foothills project, constructed in 1979-1983, includes the Strontia Springs Dam and reservoir, the Foothills Water Treatment Plant and 3.4 mile-long Foothills Tunnel that conveys raw water from the reservoir to the treatment plant. Strontia Springs Dam provides an example of recent arch dam design and construction as part of the historical development of the city of Denver and its water supply system developed by Denver Water.

Denver Water provides storage, treatment, and distribution of quality drinking water to the Denver Metro area. It is a quasi-city agency governed by a Board of Water Commissioners that consists of 5 members on staggered 6-year terms appointed by the mayor of Denver. The water supply system consists of 18 dams, both concrete and earthfill, 3 water treatment plants, 3 major tunnels (Moffat, Roberts, and Foothills), one reuse plant (in progress), and numerous pump stations and conduits.

Strontia Springs Dam is located on the South Platte River, 25 miles southwest of the city limits of Denver, in the eastern or foothills part of the Rocky Mountain Front Range of Colorado. The dam and reservoir sites lay within the complex of Precambrian metamorphic rocks (Idaho Springs Formation) that forms the core of the Front Range Uplift. The rock consists of granite gneiss, biotite gneiss, biotite schists and predominantly migmatite. The migmatite at the site is a banded, intimately-interlayered igneous and metamorphic rock containing minor amounts of pegmatite.

Strontia Springs Dam is a double curvature thin arch dam with a structural height of 299 feet, crest width of 10 feet, base width of 31 feet and backs up a 1.7 mile-long reservoir of 7,700 acre-feet of water storage (Figures 1 through 5 and Table 1). The large release facilities for Strontia Springs Dam consist of an integrated ogee service spillway near the top center of the arch dam and a fuse plug auxiliary spillway on the left abutment.

Strontia Springs Dam was designed by Harza Engineering Company, Chicago, Illinois and built by Morrison Knudsen Co., Inc., Boise, Idaho. The dam is owned and operated by Denver Water.

Strontia Springs Dam has contributed to the definition of progress in the development of dam engineering and construction technology in the 20th century. The Foothills Project was named one of the "Ten Outstanding Engineering Achievements of 1983," by the Society of Professional Engineers.

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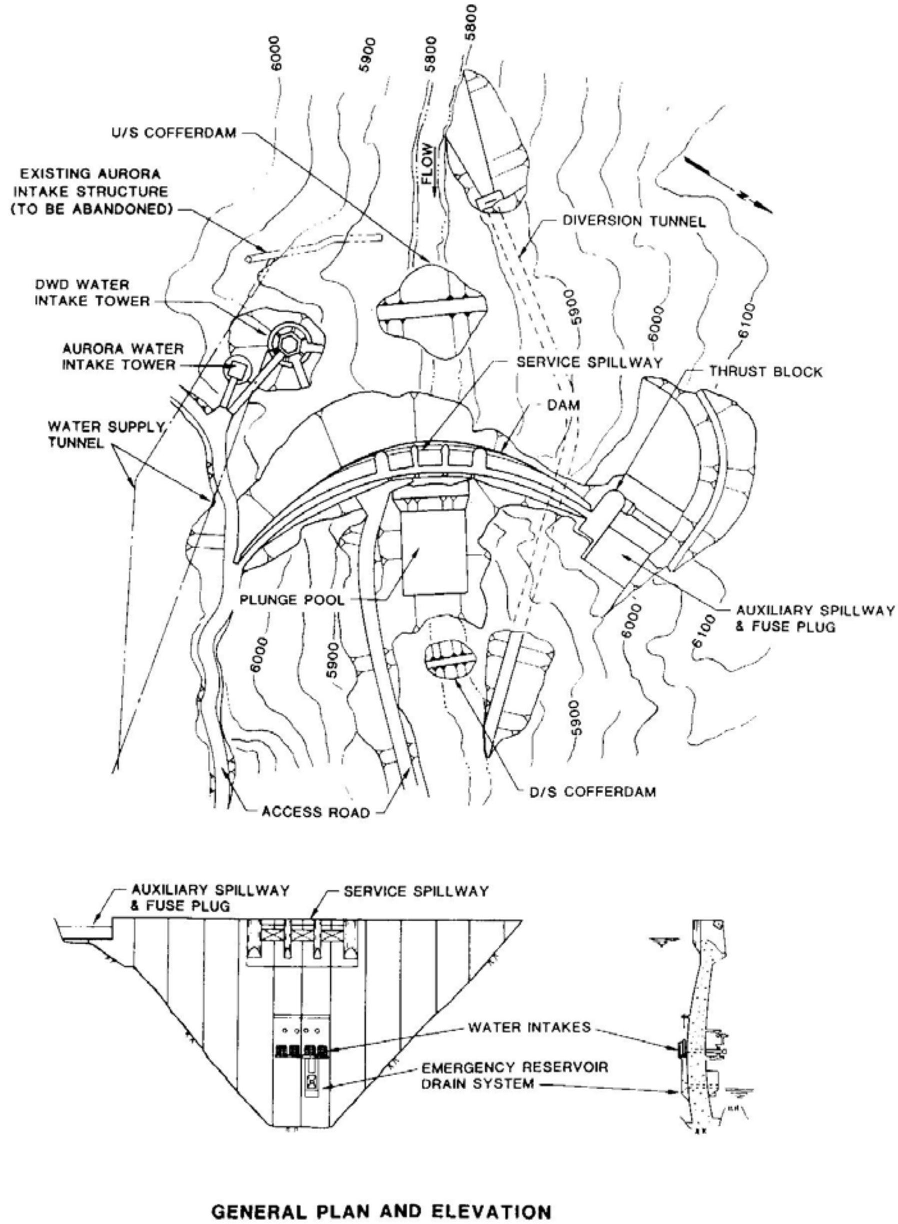

Figure 1. Plan View, Profile, and Cross Section of Strontia Springs Dam

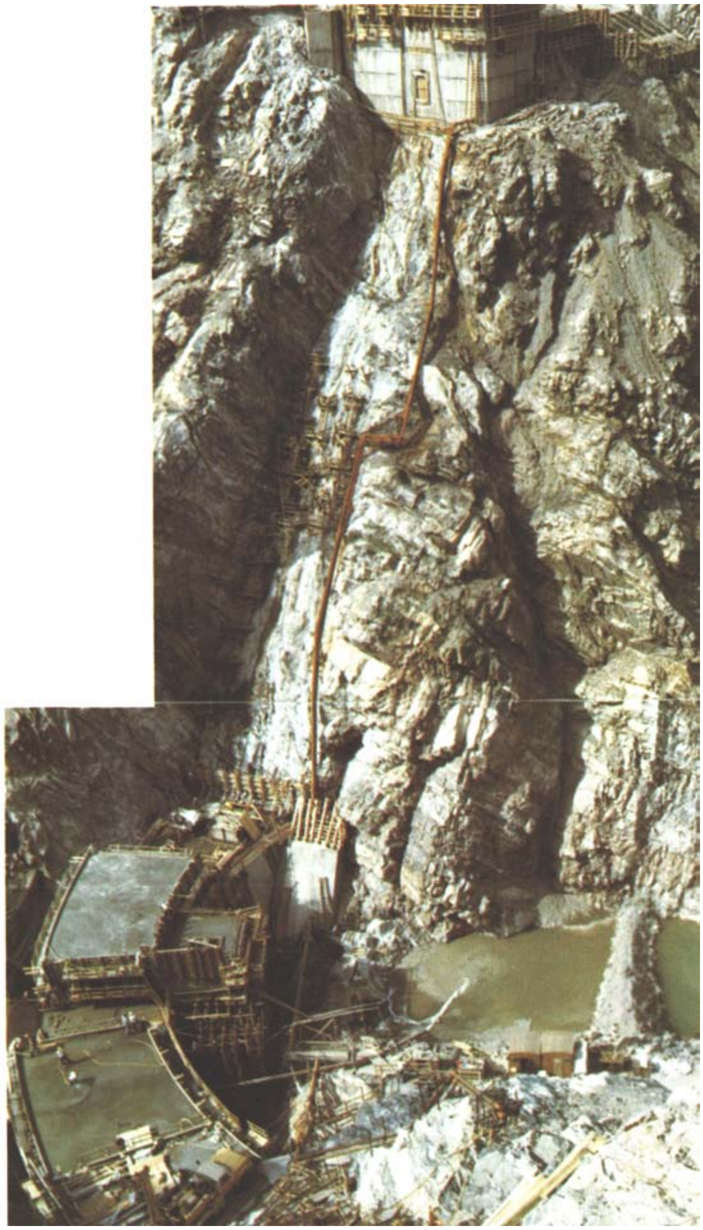

Figure 2. Excavation for Keyway on Left Abutment.

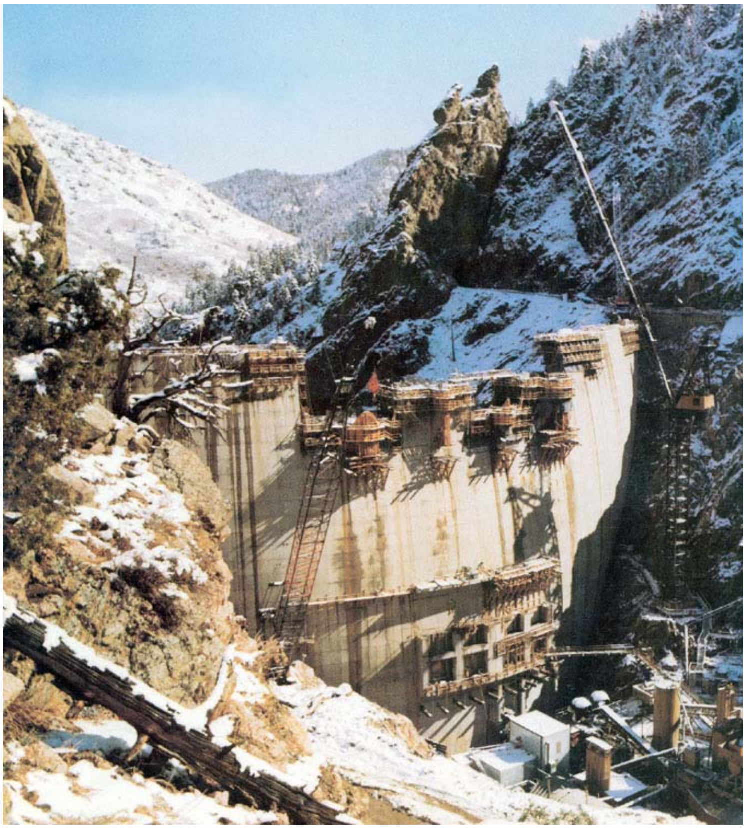

Figure 3. Winter Construction.

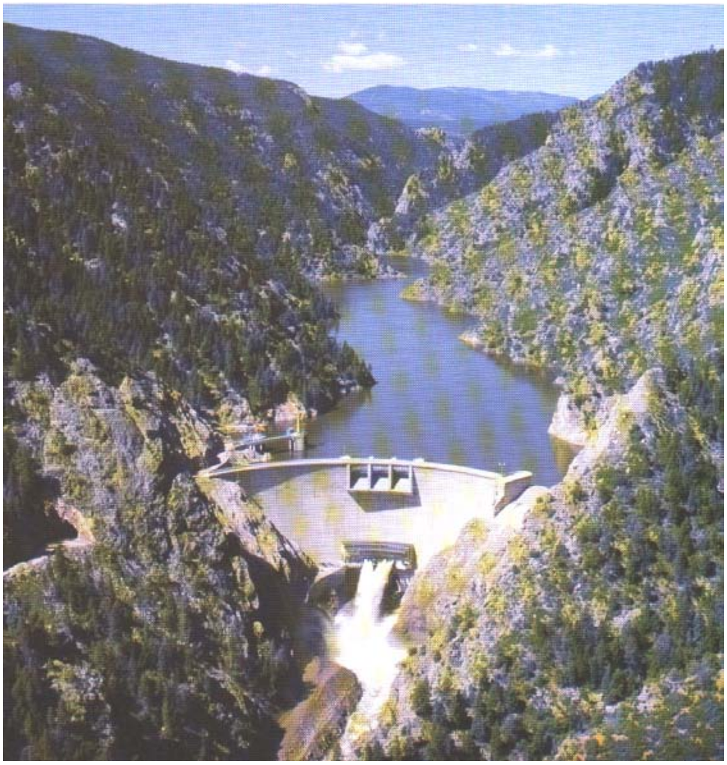

Figure 4. View of Strontia Springs Dam after Construction.

Figure 5. View of Strontia Springs Dam.

Table 1. Summary of Pertinent Information for Strontia Springs Dam

DAM	
Type	Double-curvature, thin-arch
Date Constructed	3/5/83
Concrete Volume	93,000 cy (71,000 m^3)
Maximum Structural Height	292 ft (89 m)
Height Above Streambed	239 ft (73 m)
Crest Length	650 ft (198 m)
Crest Width	10 ft (3 m)
SPILLWAY	
Type	Free overflow plus auxiliary fuse plug
Discharge Capacity	90,000 cfs (2,550 m^3s)
Maximum Recorded Flow	6,140 cfs (174 m^3/s)
MAIN WATERWAY OUTLET WORKS	
Description	Four 48-in (1 .2-m), two 18-in (0.5-m), two 8-in (0.2-m) conduits with free discharge valves
RESERVOIR	
Capacity	7,700 ac-ft (9.5 H m^3)
Surface Area/Pool Length	98 acres (40 ha)/1.7 mi (2.7 km)

MIDDLE FORK DAM

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Key Terms: roller compacted concrete (RCC), oil shale aggregate

ABSTRACT

Middle Fork Dam, constructed on Parachute Creek in 1984, was Colorado's first roller compacted concrete (RCC) dam and the first dam in which oil shale was used as concrete aggregate. This paper presents a brief history of the project including the rationale behind sizing and dam type selection, discusses the design which maximized the use of available on-site materials, and construction where the impact of appurtenant project features was minimized to enhance RCC placement. The paper also describes the methods developed by the contractor to overcome the restrictions imposed by the steep, narrow canyon at the dam site, such as building a sheet pile tieback wall which retained a temporary fill that served as a multi-purpose platform throughout construction of the dam and delivery of the RCC to the dam by conveyor including the use of an enclosed rock ladder. The paper concludes with a discussion of lessons learned during planning, design and construction related to RCC mixes, contraction joints, downstream facing, and seepage that were subsequently applied to other RCC dams around the world.

INTRODUCTION

Middle Fork Dam was the first roller compacted concrete (RCC) structure to be constructed in the state of Colorado and only the second dam of its type to be placed in operation in the United States. It is a gravity-type structure, 124-ft (38 m) high with a crest length of 420-ft (127 m) and a volume of 55,000 yd^3 (42,000 m^3). The dam is owned by Exxon Company, USA and is located on the Middle Fork of Parachute Creek at the Colony Shale Oil Project site in Garfield County, near the town of Parachute, Colorado.

The dam was designed to protect the mine facilities located downstream, from flood damage and to provide 100 acre-feet (123,400 m^3) of active storage for water supply. Exxon retained International Engineering Company, Inc. (IECO) of Denver, Colorado to design and manage construction of the project in June of 1983. Final design was completed in January 1984 and a contract for producing RCC aggregate was awarded in February of that same year. In May, the prime contract for construction of the dam was let to Avery Structures, Inc. (ASI) of Buena Vista, Colorado. Placement of RCC started at the end of July 1984 and was completed in mid-September of that same year. Work on appurtenant features of the project was completed by the end of November and initial filling of the reservoir then commenced.

PROJECT HISTORY AND RESERVOIR SIZING

In 1980 and 1981, designs were prepared for a flood control dam on the Middle Fork of Parachute Creek at the present dam site. Under an earlier design by a different firm than IECO the dam was to be an earth and rockfill structure with an intermediate level closed-conduit spillway.

Due to the potential for significant downstream damage in case of overtopping, the dam was designed to control the probable maximum flood (PMF). The control was provided primarily by storage of a high percentage of the PMF volume. The resulting design called for a dam that would have been 200-ft (61 m) high.

In the spring of 1982, construction was begun at the site, with excavation, foundation treatment activities, and fabrication of pipe, gates, and other appurtenances. However, in May 1982, construction of Middle Fork Dam was halted as part of a decision by Exxon to redirect development efforts at the Colony Project.

The estimated high cost of the dam and difficulties encountered in the higher elevations of the foundation led to a re-evaluation of the Middle Fork Dam's flood control function and size. An earth and rockfill dam would be subject to a high probability of failure if overtopped; therefore, the design would have to include a very large spillway or the reservoir would have to store almost the entire volume of the PMF. The potential failure of the embankment dam could result in a short duration, but high peak, flood wave that would equal or exceed the damage capability of floods approaching the PMF in magnitude.

A request was made of IECO to re-evaluate the project, and IECO recommended that a concrete dam be considered. The size of the concrete dam required to provide the necessary flood control was determined to be 124-ft (38 m) high, with a total storage to the crest of 390 acre-feet (481,000 m^3). The material volume of the dam section was reduced because of the decreased height and the steeper slopes, compared to the original earth and rockfill embankment design. A concrete gravity dam design required only 55,000 yd^3 (41,000 m^3) of material, compared to 650,000 yd^3 (497,000 m^3) of material in the earlier design.

Cost and schedule comparisons of various concrete dam types led to the selection of a dam to be constructed with RCC. It was estimated that an RCC dam could be constructed for significantly less cost than a conventional concrete dam and in less than six months time. Moreover, the RCC dam had the same operating features as a conventional concrete gravity dam, most importantly the ability to withstand overtopping without failure.

DESIGN FEATURES

On-Site Materials

During design of the RCC structure, consideration was given to incorporating on-site materials purchased during the previous construction effort. These included two 36-in (0.9 m) slide gates with stems and operators, and an adequate supply of 36-in-diameter (0.9 m) and 60-in-diameter (1.5 m) diameter pre-stressed concrete pipe.

Oil Shale Aggregate

Historically, concrete design practice in the region had called for processing and hauling aggregates from sources near the Colorado River, that were upstream or sufficiently downstream of tributaries which cut through the oil shale (marlstone) rich formation. It was generally accepted that concrete aggregate containing oil shale suffered a significant reduction in strength and durability as compared to concrete with aggregates derived from granite, basalt, and sandstone. Some previous tests, however, indicated that the marlstone, when crushed and graded, might prove to be a suitable aggregate material for asphalt surfacing and concrete. Therefore, in the early stages of design, crushed oil shale was used in a testing program with a variety of gradations and cement contents to evaluate its suitability as an aggregate source for RCC.

Oil shale that had been previously excavated and stockpiled was available within 1,500-ft (457.5 m) of the dam site in adequate quantities, while the nearest river gravels were approximately 20-mi (32 km) away. The economic advantages of using the local materials were therefore very prominent.

The positive compressive and tensile strength results and the apparent ease of producing and handling of the oil shale mixes were not enough to select this material as the aggregate source for the project. The aggregate's effect on the elastic and thermal characteristics of the RCC was also extremely important to the dam design. This was even more critical at the Middle Fork site, where the dam would be subjected to extremely cold ambient temperatures shortly after the interior reached its peak thermal rise, due to the heat of hydration of the cement. In order to fully assess the potential for cracking of the mass caused by this cooling, detailed adiabatic temperature rise tests were performed and tests were made of the thermal coefficient of expansion, modulus of elasticity, and modulus of rupture.

Important properties of the RCC using oil shale as aggregate were as follows:

Specific Heat	0.22 BTU/lb-^{o}F (0.48 BTU/kg-^{o}C)
Thermal Diffusivity	0.027 ft^2/hr (0.29 m^2/hr)
Thermal Conductivity	0.80 BTU/ft-hr-^{o}F (2.6 m-hr-^{o}C)
Coefficient of Thermal Expansion	0.0000035/^{o}F (0.0000027/^{o}C)
Poisson's Ratio (28 days)	0.16
Adiabatic Heat Rise (28 days)	260^{o}F (127^{o}C)
Compressive Strength (28 days)	1,120 lbs/in^2 (78.7 kg/cm^2)
Direct Tensile Strength (28 days)	190 lbs/in^2 (13.4 kg/cm^2)
Modulus of Elasticity (28 days) (static)	980,000 lbs/in^2 (6,757 Mpa)
(sustained load 28 to 365 days)	710,000 lbs/in^2 (4,895 kg/cm^2)
Stress Coefficient	2.5 lbs/in^2/ft (0.6 kg/cm^2/m)

Internal dam temperature changes with time were simulated by a computer model that utilized over 1,000 points within the dam cross section to calculate temperatures as the dam was theoretically constructed, lift by lift. In the model, the mass was subjected to heat rise due to cement hydration and then cooled when exposed to the atmosphere. Various rates of construction, lift thicknesses, and starting dates were examined to determine the primary sensitivity to the various factors. The time of year of placement, the heat rise due to cement content, and the mass's diffusivity were the most critical parameters.

Compared to other materials used in mass concrete, the Middle Fork RCC dam with oil shale aggregate resulted in a low thermal stress per degree of temperature drop. The stress coefficient was less than one-half that computed for Willow Creek Dam constructed by the U.S. Corps of Engineers in 1981, the only other operational RCC dam in the United States at that time.

These very positive characteristics of the RCC using oil shale aggregates resulted in not having to place any restrictions on rates of RCC placement. This, along with availability of the aggregate at the site, generated substantial cost savings for the owner.

Simplification of Construction

RCC is a zero-slump mass concrete placed and compacted with earthmoving equipment. Its construction-related economic advantages result from being able to build the dam in a short time, monolithically and in a continuous operation without interruptions. Forming is reduced by eliminating transverse control joints, and cement content is kept to a minimum (only 112 lbs/yd^3 [67 kg/m^3] at Middle Fork). In order to maximize the savings from these construction advantages, design emphasis was placed on simplifying the appurtenant features as much as possible and avoiding interference with RCC placement. The general features of the dam and appurtenances and their design are shown in Figure 1 and are described as follows.

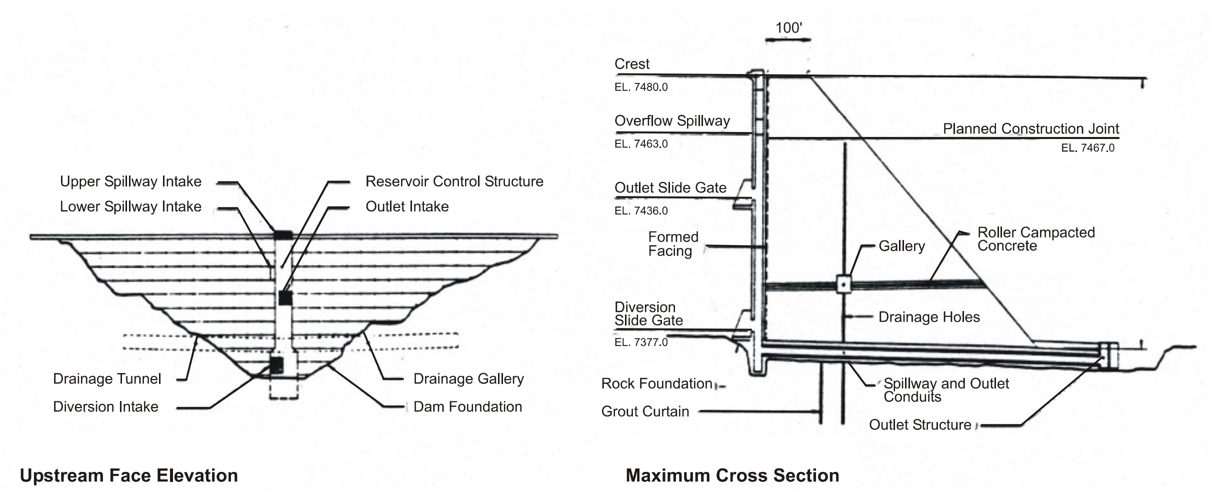

Figure 1. Upstream Elevation and Maximum Cross Section of Middle Fork Dam.

The primary spillway and the outlet works were combined in a double-chambered tower placed against the upstream face and connected to conduits in a trench underneath the dam that led to an outlet structure downstream of the dam. This enabled the conduits to be installed prior to the start of RCC placement and allowed the tower and outlet structure to be constructed independent of the main body of the dam, thus avoiding any constraints to RCC placement. There are three inlets, one gated and two ungated, into the 60-in-diameter (1,500 mm) diameter spillway conduit, and one gated inlet for control of the 36-in-diameter (900 mm) diameter outlet works which is used to regulate active storage. The spillway and reservoir flood storage will handle a 500-year flood event before overtopping. The dam will safely tolerate the overflow of a PMF.

The dam was founded on sound oil-rich marlstone near the top of the Parachute Creek member of the Green River Formation, and near the base of the Uinta Formation, in the core trench excavated during the previous embankment dam construction. A grout curtain with a maximum depth of 90-feet (27.5m) had also been constructed at that time. Grouting of the near-surface rock was planned following construction of the dam, if required. Foundation treatment therefore was limited to removal of loose rock and final cleanup. Major irregularities in the foundation surface were backfilled with dental concrete prior to placement of the RCC. A construction joint was planned at about two-thirds the dam height, just below a zone of weathered rock on the abutments. This pause in RCC placement allowed the contractor to complete upper level excavation and foundation cleanup using mechanical equipment. The design also called for drilling drain holes through the dam and into the foundation.

A gallery for inspection and drainage was constructed parallel to the dam axis just downstream of the grout curtain. The center portion of the gallery was formed within the dam section and represented the only obstacle to the desired rapid placement of RCC during construction. The drainage gallery was extended horizontally, as unlined tunnels, excavated into the abutments at the time of foundation cleanup. The gallery access adit was located at the dam/abutment interface on a rock ledge. Both of these features minimized interference with RCC placement.

Seepage Control Measures

Excessive seepage through the RCC construction joints was experienced at the previously mentioned Willow Creek Dam. Therefore, measures were taken to reduce seepage through Middle Fork Dam and to control any that might occur. The principal means of accomplishing this was to specify tight control over the RCC placement itself. Due to the narrow canyon and relatively small volumes required for each lift, it was possible to minimize the elapsed time between lifts and thus ensure a good bond. In addition, a great deal of attention was given in the specifications to reducing segregation and contamination of the large-size aggregate (4-in [100 mm] maximum) while spreading the RCC during construction. A vertical upstream face of conventional air-entrained concrete, one-ft-thick (300 mm), utilizing aggregates from the Colorado River which met ASTM C-33 requirements, was specified. This facing provided a barrier against seepage, as well as protection of the upstream face of the dam against freeze-thaw action. A two-in (50 mm) layer of bedding concrete was placed immediately downstream of the facing concrete for a distance of six-ft (1.8 m). Bedding concrete differed from facing concrete in that it utilized fine RCC aggregate. It was placed between each RCC lift to incorporate any aggregate that might segregate at the bottom of a lift. As illustrated in Figure 2, the sequence required placing the facing and bedding first (steps 1 and 2), then spreading and compacting the RCC into them (steps 3 and 4); thus ensuring a tight bond between the two lifts.

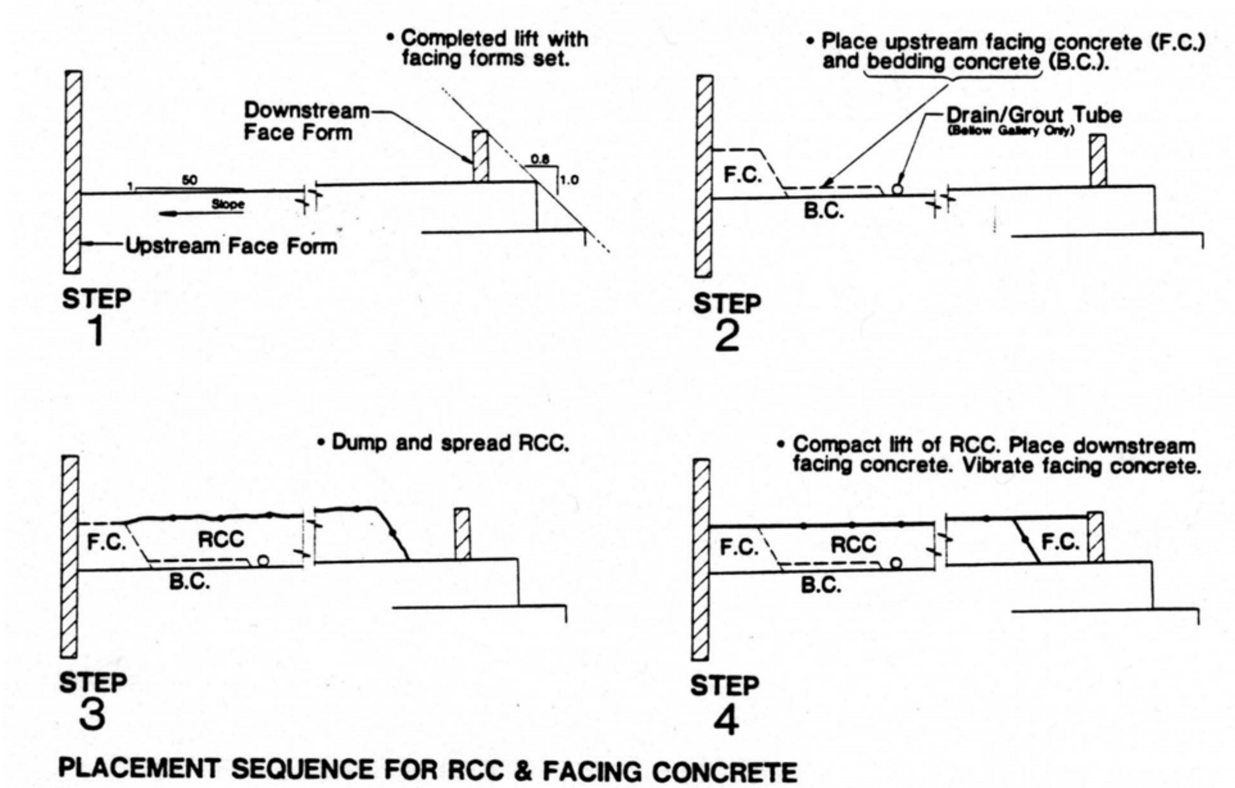

Figure 2. Placement Sequence for RCC and Facing Concrete.

The upstream facing was provided with horizontal rustication grooves at one-ft (300 mm) intervals to mask the joints between lifts. In addition, 3-in (76 mm) deep vertical grooves on 12-ft (3.6 m) centers were formed in the facing to control the location of shrinkage cracks. These vertical grooves were sealed with an elastometric caulk after cracks developed following completion of the dam. Geofabric drain/grout tubes were installed between the first 18 lifts of the dam just downstream of the bedding concrete in order to collect any seepage developing through lift contacts that might occur. After filling the reservoir, if this seepage is deemed excessive, the tubes would then be used to introduce chemical grout into the RCC lift joints.

A drainage system was provided within the dam to control any seepage that might occur. It consists of 3-in (76 mm) diameter holes drilled at 10-ft (3 m) centers through the dam, from the normal pool elevation to the drainage gallery. These holes were extended into the foundation to varying depths to relieve uplift pressures.

Downstream Face

The 0.8H to 1.0V downstream face of the dam was formed and constructed with conventional concrete, identical in mix design to the upstream facing, in 1-ft (300 mm) high steps following each RCC lift in the dam. This facing provided several advantages over an unformed RCC slope, such as reducing material waste over the side, freeze-thaw resistance, superior wearing surface in the event of overtopping, support for a safety barrier during construction; and an aesthetically pleasing appearance.

AGGREGATE PRODUCTION AND HANDLING

The construction schedule called for crushing the oil shale (marlstone) stockpiled about 1,500-ft (457.5 m) downstream of the dam into aggregate during the late winter and early spring and to haul it up through the narrow canyon and re-stockpile it upstream of the dam. The intent was to remove this work item from the critical path of the dam construction and to pre-cool the aggregate by stockpiling it during the colder months. Specifications called for producing three aggregate sizes, 4-in (100 mm), to 1 ½-in (38 mm), 1 ½-in (38 mm) to #4 (4.75mm), and #4 (4.75mm) minus. Because of the flat, elongated nature of the oil shale fragments, an impact type crusher was specified at some point in the plant layout in order to achieve better grain shape distribution. Problems were encountered in screening out the #4 size (4.75mm) due to freezing and binding of the wet fine material on the screen. The specification was changed to make two aggregate sizes conforming to the following gradations:

Coarse RCC Aggregate

Sieve Size	% Passing
4-inch (100mm)	100
3-inch (76mm)	90-100
2-inch (50mm)	65-90
1-1/2-inch (38mm)	30-80
3/4-inch (19mm)	0-20
1/2-inch (13mm)	0-10

Fine RCC Aggregate

Sieve Size	% Passing
1-1/2-inch (38mm)	100
3/4-inch (19mm)	82-100
1/2-inch (13mm)	70-91
3/8-inch (9.5mm)	57-85
#4 (4.75mm)	46-67
#8 (2.36mm)	25-55
#16 (1.18mm)	15-47
#30 (600mic)	8-38
#50 (300mic)	5-30
#100 (150mic)	5-23
#200 (75mic)	2-17

56,000 tons (50,960 metric tons) of coarse aggregate and 66,000 tons (60,060 metric tons) of fine aggregate were crushed and hauled before mid-May 1984, which did not interfere with mobilization for dam construction. These volumes included a 20 percent contingency to cover uncertainties in actual quantity requirements.

DAM CONSTRUCTION

Work Schedule

Following notice-to-proceed on May 7, 1984, the contractor worked a single 10-hour shift, six days per week during the site preparation work until July 29. A swing shift was added for foundation treatment on July 16. At the start of RCC placement on July 30, two full 10-hour shifts were initiated, seven days per week, and continued through the end of RCC placement on

September 15. The shifts ran from 4:00 a.m. to 2:30 p.m. and 2:30 p.m. to 1:00 a.m. The three-hour period between shifts was utilized for refueling and equipment maintenance.

Site Preparation

Construction activities, preparatory to the start of RCC placement in the dam continued through the months of May, June, and July. These activities consisted of diverting the creek, building the permanent dam access roads, driving the drainage tunnels in each abutment, foundation treatment, and excavation in the valley floor. The trench for the spillway and outlet conduits was excavated in the rock in the canyon floor. The conduits were laid and backfilled with concrete. Foundation preparation consisted of dental excavation, dental concrete, and cleanup of the rock surface with air and water jets. The reservoir control tower on the upstream side of the dam was constructed to about 15-ft (5 m) above the dam foundation. This enabled the diversion slide gate to be installed. The outlet structure was completed at the downstream toe of the dam and second stage diversion was initiated prior to the start of dam construction.

The contractor constructed a temporary sheet-pile tieback wall that retained a temporary fill parallel to, and about 20-ft (6 m) upstream of, the upstream face of the dam. The wall consisted of 50-ft (23 m) (maximum) high sheets, held in place by "I" beam walers and rebar tiebacks anchored into the fill by concrete "dead-men", as shown on Figure 3. The valley upstream of the wall was backfilled with required excavation material to elevation 7430 (2,266 m), or about mid-height of the dam. This temporary platform served as a working area throughout construction of the dam and formed the foundation for the RCC batch plant, conveyor system, and platform from which a large crane operated. Figure 4 shows the construction site layout in plan view.

Figure 3. Core trench excavation for the original dam from the right abutment. Note temporary construction tie-back retaining wall and work area on the left.

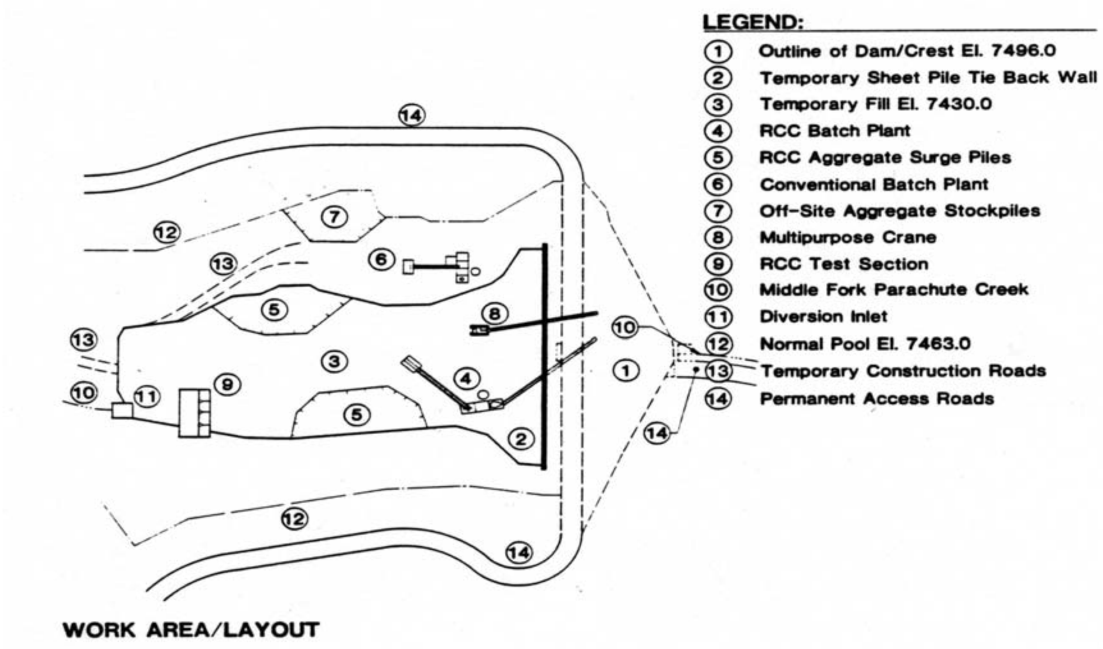

Figure 4. Work Area/Layout.

A 200 y^3/hr (153 m^3/hr) batch plant for RCC and a 100 y^3/hr (77 m^3/hr) batch plant for conventional concrete were mobilized as soon as the temporary fill was completed in early July 1984. The RCC plant was situated on the temporary fill about 25-ft (8 m) upstream of the sheet-pile retaining wall. The conventional plant was located on a rock bench on the left abutment, slightly farther upstream than the RCC plant.

RCC Mix Design and Field Test Section

In mid-July, after the batch plants had been erected, an RCC test section was constructed just upstream of the dam. The objective of the test fill was to reproduce actual dam placement conditions so that both the contractor and the field engineer could gain experience with handling and placing RCC. Nine lifts were placed in three days in the initial test section, which measured approximately 20-ft (6 m) by 40-ft (12 m).

The RCC mix design per cubic yard using the saturated surface dry (SSD) weights, as determined from laboratory testing and the test fill, was as follows:

1,316 lb (790 kg/m^3) Coarse RCC Aggregate
1,974 lb (1,184 kg/m^3) Fine RCC Aggregate
112 lb (67 kg/m^3) Cement
160 lb (96 kg/m^3) Water

During the test fill and the first few days of placement on the dam, mixer efficiency tests were performed on the RCC. This test consisted of determining a coefficient of variability based upon the percentage to which the cement, water, and aggregate were distributed through the mix as compared to the theoretical batch proportions. The intent of these tests was to reduce mixing time in the batch plant to a minimum and at the same time to ensure an adequate dissemination of the material components throughout the mix.

RCC Delivery System and Equipment

The contractor's system for delivering RCC from the Ross batch plant, located on the temporary fill, to the lower elevations of the dam consisted of a gob hopper, which discharged onto a stacker conveyor and then into a rock ladder. This rock ladder consisted of a 36-in (0.9 m) diameter, steel pipe with baffles every 5-ft (1.5 m). This minimized segregation of the RCC mix due to the greater than 50-ft (15 m) vertical drop to the construction surface. As the dam rose in elevation, sections of the rock ladder were progressively cut off and the vertical drop decreased. When the dam reached mid-height, elevation 7430 (2,252 m), the rock ladder was removed and the conveyor discharged RCC directly onto the dam.

For the upper part of the dam above the planned construction joint, three conveyors were utilized in a scissored arrangement, with the last stacker conveyor mounted atop a 45-ft-high (13.6 m) high tower made of scaffolding. After the RCC reached the dam from the conveyor, it was either dumped directly onto the dam surface and picked up with a front-end loader, or caught by the loader. Near the crest of the dam, when the fill width became very narrow, the conveyor discharged directly into a 10-y^3 (7.6 m^3) dump truck and was shuttled back and forth, parallel to the crest. The loader and truck only operated on previously compacted RCC surfaces. Figure 5 illustrates the changes in the delivery system as the dam rose.

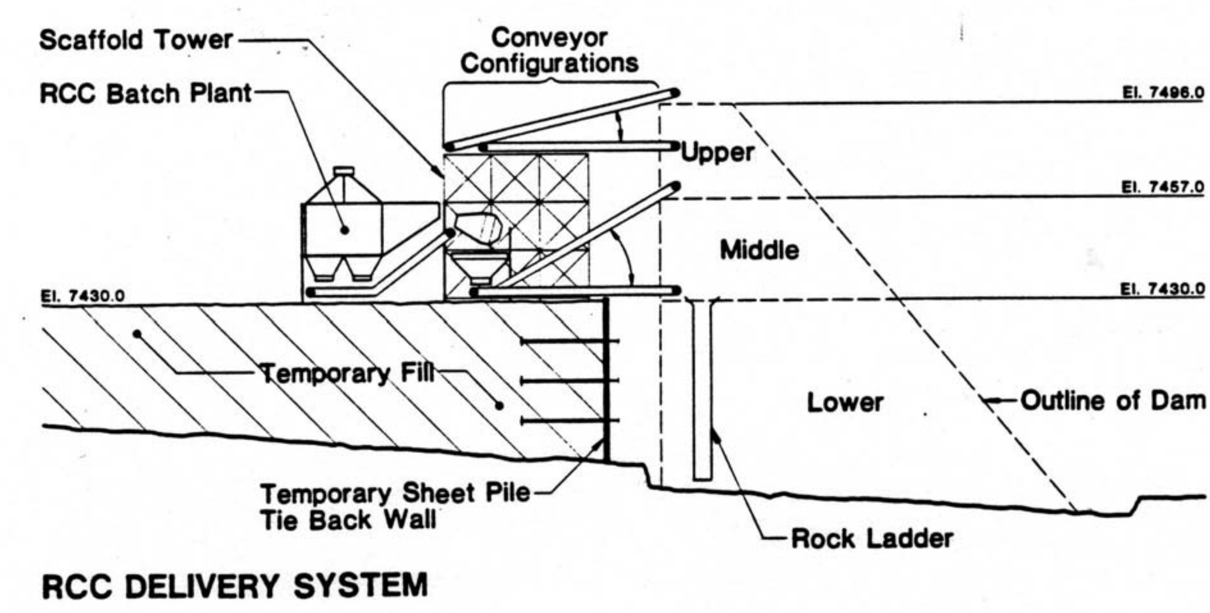

Figure 5. RCC Delivery System.

From the conveyor discharge surge pile, the RCC was picked up with a front-end loader, shuttled to the desired placement point, and deposited in piles. The piles were knocked down and spread in even one-foot (300mm) lifts with a bulldozer. The bulldozer worked in a back-and-forth direction, parallel to placement lanes, and took care not to turn on the uncompacted RCC to minimize segregation by working the material toward the center of the lane with the blade. The RCC was then compacted with either a single or double-drum vibratory roller. A crew of manual laborers accompanied the operation to shovel any segregated coarse aggregate back into the fill, to keep the fill moist, to compact with whackers and small rollers along the abutments and facing, and to place and consolidate the conventional bedding and facing concrete.

RCC Construction

Construction of the RCC dam began on the afternoon of July 30 1984. A thin layer of about 2-in (50 mm) of bedding concrete was placed over the rock foundation for the initial coverage. For the lower elevations, the RCC was spread and compacted in an upstream/downstream direction due to the narrowness of the canyon (40-ft [12 m] wide x 1,301-ft [394 m] long) with a negative 5-percent slope toward the upstream face. This practice continued until the drainage gallery invert elevation of 7388 (2,239 m) was reached. At that time, the traffic was routed parallel to the dam axis and the slope was reduced to 2-percent; this practice continued through completion.

A typical lift sequence (shown in Figure 2) consisted of placing RCC with the loader at the downstream right corner of the dam surface. After a sufficient volume had been deposited, the RCC was spread to a thickness of about 13-in (330 mm) with a bulldozer. Grade and lift thickness was controlled with a laser, rustication strips on the upstream face forms, and numbered lift boundaries painted on the abutments. The RCC was spread in four or less lanes about 20-ft (6 m) wide, parallel to the dam axis above the roof of the gallery. As the dam section became narrower, the number of lanes was reduced accordingly. The RCC placement surface was kept damp at all times by a laborer, spraying with a hose.

Concurrent with the start of RCC placement, conventional bedding concrete was placed about two inches thick against both abutments. Simultaneous with the spreading of the downstream lane, RCC was spread into the fresh bedding against the abutments. The loader that delivered the RCC was careful to operate only on the previously compacted RCC surface. After a lane advanced a sufficient distance, the RCC was rolled with from 8 to 12 passes of the vibratory roller. Generally, elapsed time between one lane and another was about one hour. As the RCC advanced toward the upstream face, facing concrete was placed against conventional cantilevered tie-back steel forms to a width of one-ft (300 mm) at the top and two-ft (600 mm) at the base; followed by placement of bedding concrete in a thin one-in (25 mm) to two-in (50mm) layer for a distance of six-ft (1.8 m) downstream from the facing.

The facing concrete was structural, air-entrained concrete with a water/cement ratio of 0.44 and a slump of two to three-in (50-76 mm) using off-site conventional aggregate. The bedding concrete was conventional concrete of higher slump using either off-site or oil shale aggregate. Above El. 7470 (2,264 m), the six-ft (1.8 m) width of bedding was reduced to two-ft (600 mm). As the RCC advanced, it was dozed into the facing concrete. The facing concrete was then vibrated internally prior to compaction of the adjacent RCC. The RCC was rolled as close to the upstream face as the physical constraints of the roller and safety would allow, usually one or two-ft (600 mm) from the face. The RCC/facing concrete interface was then compacted with a smaller roller or whacker; as was the RCC and bedding concrete near the abutments. The last step in the lift sequence was to place the downstream facing concrete. (identical to the upstream facing in mix design) between the one-ft (300 mm) high steel forms and the RCC and vibrate it internally. At times, the downstream lagged a maximum of two lifts behind the RCC.

Production of RCC averaged four lifts per day with only a few delays. Work on the RCC dam was halted at elevation 7457.5 (2,260 m) from August 26 to September 4 to allow for drilling of the internal drain holes through the dam and for completing excavation and cleanup of the abutments from elevation 7460 (2,260 m) to 7496 (2,272 m). Construction joints in the RCC 10 hours or more old were treated by cleaning up the surface and covering with a thin layer (one- in [25 mm]) of bedding concrete. Construction of the RCC dam was completed in the early morning of September 15, 1984. Figures 6, 7, and 8 show typical placement conditions at the base, mid-height, and near the crest of the dam respectively.

Figure 6. Night-time RCC placement operation at lower elevation of the dam as viewed from upstream on the right abutment.

PERFORMANCE

Following completion, the dam was filled to the intake, level 44-ft (13 m), above the reservoir floor in the fall of 1984. Total seepage of about 150 gallons-per-minute (gpm) (9.5 l/s) was recorded at a v-notch weir installed in the gutter at the entrance to the drainage gallery access adit. Total seepage decreased to about 110 gpm (6.9 l /s) after two and one-half months. The intake gate was closed in the spring of 1985 and the reservoir rose to a height of 87-ft (26.5 m), corresponding to the primary spillway elevation. Seepage increased to 475 gpm (29.9 l/s). Eighteen months later seepage had decreased to less than 48 gpm (3 l/s). The seepage reduction occurred primarily due to calcification of the mass and was most pronounced over the first three months, with the rate of healing decreasing with time. Initially, 80 percent of the total seepage came from within the dam via floor drains and roof drains, and the walls of the gallery. The remainder emerged from the drainage tunnels in each abutment. This contribution from the dam

Figure 7. RCC dam at about two-thirds height with conveyor delivering RCC directly onto the lift surface.

Figure 8. RCC placement in the upper part of the dam as viewed from downstream high on the right abutment. The work area with batch plants, conveyors, and crane can be seen at the left. The complete RCC placement sequence of delivery, spreading, and compaction can be viewed. Note the stepped downstream facing and safety barriers.

proper dropped to about 50 percent as the dam healed. The foundation flows remained nearly constant. Approximately 25 percent of the seepage came from the internal drainage curtain. Calcium carbonate precipitated at the point of emergence of the seepage within the gallery. The drains required periodic cleaning. Seepage volume from the dam was judged acceptable by the owner, considering the functional requirements of the structure. The dam has continued to perform satisfactorily since. Figure 9 shows the completed dam from downstream and upstream on the right abutment.

Figure 9. Completed dam as viewed from downstream and upstream on the right abutment. The reservoir control structure can be seen rising up the face of the dam. Note the vertical crack-control grooves and horizontal rustication grooves in the upstream face, and the steps on the downstream face.

CONCLUSIONS

As Colorado's first RCC dam, and the first concrete dam in the state to be constructed using oil shale as aggregate, Middle Fork Dam, is an important monument to the history of engineering geology in the state. In addition to these firsts, several other RCC dam design features introduced at Middle Fork Dam such as crack inducers and seals at the upstream face, drain/grout tubes, internal drainage, and formed downstream steps of conventional concrete, were subsequently used on many RCC dams around the world. The project also demonstrated that by applying innovative construction techniques, the use of RCC could allow a dam to be constructed in a single construction season in the harsh mountain climate of Colorado with minimal impact on the environment.

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SPRING CREEK DAM MODIFICATIONS

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Key Terms: embankment dam, inadequate spillway, roller compacted concrete (RCC), probable maximum flood (PMF)

ABSTRACT

Spring Creek Dam was the first earth embankment dam in Colorado, with inadequate spillway capacity, to be modified utilizing roller compacted concrete (RCC). With a structural height of 50-ft (15 m), it was at the time (1986) the highest dam in the world to be rehabilitated using this economically attractive and environmentally pleasing solution. The RCC lining was designed to safely pass the probable maximum flood (PMF), and was completed at approximately one third the cost of previously considered alternatives. The technology pioneered at Spring Creek Dam proved to be an economical solution to the inadequate spillway problem that has subsequently been applied at many high hazard dams in Colorado and throughout the United States.

This paper is a case history that describes the project background and deficiencies. It goes on to discuss the conceptual rehabilitation plan, RCC trial mix program, hydraulic model study, and final design. The paper concludes with a description of the construction of the RCC spillway.

INTRODUCTION

During the Phase I inspection of the National Dam Safety Program (NDSP), sponsored by the U.S. Army Corps of Engineers (USACE), approximately 16 percent of the high hazard dams in Colorado were found to have insufficient spillway capacity to pass the probable maximum flood (PMF). Many of these dams are located in high mountainous regions where access is difficult, the construction season is short, and material availability is limited.

Spring Creek Dam is one of these dams (Figure 1). It is an earth embankment dam approximately 50-ft (15 m) high with a crest length of about 575-ft (174 m). The dam crest is situated at elevation 10,022-ft (3,037 m) above mean sea level (msl) and is located approximately 25-mi (40 km) northwest of the town of Gunnison. A service spillway is located on the right abutment, and is a broad crested concrete weir, with a concrete lined chute terminating in a stilling basin at the toe of the dam. The outlet works consists of a 18-in-diameter concrete pipe, controlled at the upstream end by an inclined slide gate. The dam was constructed in 1961-62 by the Colorado Division of Wildlife (CDOW) and the reservoir is maintained for recreational purposes.

Figure 1. View of Spring Creek Dam and service spillway prior to modification. Photograph taken from the right abutment looking upstream.

GEOLOGIC SETTING

Spring Creek Dam is located on Spring Creek, a tributary to the Taylor River, which is a tributary to the Gunnison River. Spring Creek flows through a broad glaciated valley incised into Paleozoic rocks of the Minturn Formation (Pennsylvanian), Leadville Limestone (Mississippian), and Sawatch Quartzite (Cambrian). These units overlie undifferentiated Pre-Cambrian crystalline rocks. The valley is characterized by gentle lower slopes composed of talus derived from sub-vertical limestone cliffs part way up the valley walls.

Spring Creek Dam is located in a narrow notch in a terminal moraine at the end of the upper valley. The right abutment of the dam is formed by relatively flat glacial till. The left abutment is steeper and is characterized by weathered crystalline bedrock partially masked by colluvium. The river bed below the dam is composed of well graded and rounded sand and gravel.

PRELIMINARY STUDIES

Engineering Design

Engineering design of the modifications for Spring Creek Dam was carried out in two phases by Morrison Knudsen Engineers, Inc. under a contract to the CDOW. The first phase called for developing a number of potentially feasible alternatives that would allow the dam to successfully handle the PMF without failure. The alternatives considered were:

- Construction of a large auxiliary spillway on one of the dam abutments
- Raising the dam
- Providing overtopping protection of the downstream slope with RCC

Preliminary designs and cost estimates were prepared for each alternative. The alternatives were compared considering a number of variables including constructability, construction cost, and environmental impacts.

The RCC overtopping protection alternative was judged to be the most feasible option as it offered rapid construction, at the least price and with minimal impacts to the environment.

Geotechnical Investigations

Geotechnical investigations consisted of field explorations and laboratory testing. A number of boreholes were drilled through the dam and into the foundation, in the abutments, and downstream of the dam. The boreholes were drilled to characterize the nature of the soils and rock making up the dam as well as the foundation and abutments. Boreholes were supplemented by backhoe pits in the area of potential construction materials.

Construction Materials

Three potential sources of construction materials for RCC aggregate and underdrain material were identified and characterized in the laboratory in terms of gradation, Atterberg limits, density, absorption, and resistance to abrasion. These materials can be summarized as follows:

- Colluvial deposits remaining from a borrow pit used during the original construction of the dam, located on the left abutment just upstream of the dam axis. The material is characterized by hard angular fragments of limestone, in a matrix of low plasticity, well graded silty sand, with approximately 20 percent finer than the #200 sieve.

- Glacial till from the terminal moraine that forms the right abutment of the dam. This material consists of silty sandy gravel with boulders to several feet in diameter. Gravel is sub-rounded to angular originating from crystalline, quartzite and limestone formations upstream.

- Alluvial sand and gravel in Spring Creek is well rounded, sound, well graded, with a lithology similar to the glacial till. However, the material occurs in a narrow band adjacent to the stream bed, and exploration would have had a severe impact on riparian habitat for a considerable distance downstream of the dam.

RCC Mix Design

Roller compacted concrete (RCC) consists of zero slump concrete with a low-cement content. The RCC is mixed in either batch or pug-mill type concrete plants and then delivered to the placement surface by any convenient method. The material is then spread in layers and compacted with a smoothed drum vibratory roller. The rapid placement without the need for form work allows RCC to be economically used in concrete gravity dams, paving and numerous other applications.

The RCC mix for the Spring Creek Dam, downstream facing was designed primarily to resist erosion during overtopping and to be durable under freeze-thaw conditions. Cracking due to thermal stresses, that can be a concern on large RCC dams, was not a concern for this application. A series of trial mixes, using a variety of cement contents and the locally available aggregate sources described above, indicated that RCC with acceptable characteristics could be made. Based on these test results and durability experience with soil cement, a cement content of 265 lbs/yd^3 (158 kg/m^3) was selected using the colluvial material. The material was processed to a maximum aggregate size of 2-in. The RCC aggregate had approximately 50 percent and 7 percent passing the #4 and #200 sieves respectively.

DESIGN

Flood Routing

The modified Spring Creek Dam was designed to safely pass the PMF by converting the entire downstream slope of the dam into an RCC lined auxiliary spillway. The original spillway could pass approximately 910 ft^3/s (26 m^3/s) before the dam would overtop, while the peak discharge of the PMF was calculated to be approximately 19,800 ft^3/s (571m^3/s). With the modified dam, the depth of flow in the auxiliary overflow section would be a maximum of 4.6-ft (1.4 m) near the crest and would be contained by lateral RCC walls. The maximum duration of overflow was computed to be five hours.

Downstream Apron

The stream channel for a distance of 66-ft (20 m) downstream of the toe of the modified dam was stripped of vegetation and an apron was constructed. The 26-ft (8 m) immediately downstream of the toe was blanketed with 3-ft (0.9 m) of processed free-draining material and overlain by a 3-ft-thick (0.9 m) slab of RCC sloping 1.8 percent downstream. The slab is anchored at the downstream end with a 5-ft-deep (1.5 m) by 6-ft-wide (1.8 m) RCC keyway which serves to resist undermining of the slab. The final 40-ft (12 m) of the apron transitions into the natural stream bed and is protected with riprap derived from required excavation at the base of the left abutment slope. The apron functions for both the service and the auxiliary spillways, as well as for discharges from the modified outlet works conduit.

Outlet Works Extension

The original outlet works conduit was extended 50-ft (15 m) downstream, to the toe of the modified dam, with an 18-in-diameter (200mm) polyvinyl chloride (PVC) pipe. The pipe emerges at the toe of the sloping section of the RCC lined auxiliary spillway, and discharges onto the RCC apron. Since the slope of the apron corresponds to the approximate existing stream channel gradient, no further energy dissipation was required.

Modified Downstream Slope Lining

The downstream face of the existing dam and the right abutment to the service spillway was stripped, trimmed, filled, and dressed to a slope of 2.3H to 1V from the crest, to a point 40-ft (12 m) downstream of the original dam axis (Figure 2). At this point, the slope was flattened to 3H to 1V to 145-ft (50 m) downstream of the axis. The flattening of the existing dam slope was performed to accommodate the excavated material from the abutments and produce an auxiliary spillway chute as wide and as uniform as possible. The entire downstream slope is covered with RCC approximately 3-ft (0.9 m) thick. The RCC was placed in 12-in-thick (300 mm) horizontal lifts starting at the base of the slope. The resulting steps across the surface of the RCC liner can aid in dissipating energy in case of overtopping. The lateral containment walls were constructed against the cut slope on the left side of the apron and on the right side of the existing service spillway. The height of the walls was based on the calculated flow depth and the results of hydraulic model testing.

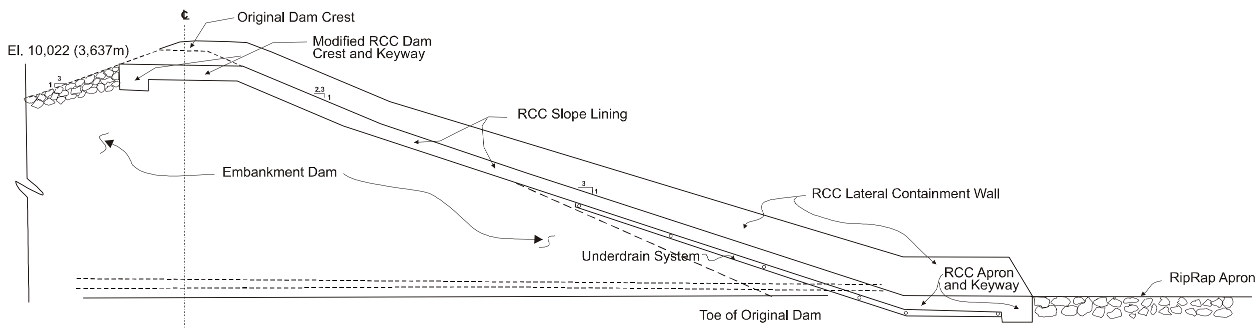

Figure 2. Modifications to downstream slope of the dam. Maximum cross section.

Underdrain System

The lower part of the sloping RCC-lined section from 80-to-145-ft (24-to-50 m) downstream of the axis was underlain by a 12-in-thick (300 mm) layer of processed free draining material. Perforated PVC pipes were embedded in the free-draining material underlying the RCC apron slab, and continue up the edge of the containment walls to collect seepage and discharge it at measuring points downstream of the RCC apron. In addition, a drain system was provided to the right of the service spillway and along the stilling basin wall to capture and measure existing seepage. Three-in-diameter (76 mm) weep drain pipes were also provided through the RCC slab at 10-ft (3 m) centers at several elevations to provide relief from uplift pressures.

Modified Dam Crest

The top 3-ft (0.9 m) of the existing dam crest in the 250-ft (76 m) wide overflow section was removed and replaced with RCC across the full length of the crest. The crest slab was anchored at the extreme upstream end by a 5-ft (1.5 m) deep by 6-ft (1.8 m) wide RCC cut-off key. The crest was provided with lateral RCC walls, sized to safely pass the design flood.

The original dam crest outside the limits of the new auxiliary overflow section and the original service spillway was raised utilizing impervious fill material derived from excavation of the overflow crest and the borrow area. The raised crest is 12-ft-wide (3.7 m) and is sloped 2 percent upstream to drain. Minimum freeboard during the 500-year flood is 3-ft (0.9 m), and 1.3-ft (400 mm) during the PMF.

CONSTRUCTION

During the month prior to the start of RCC placement operations, the contractor performed various preparatory tasks. The downstream face of the dam was shaped, drainage blanket material and perforated under drain pipes were installed, and the outlet conduit was extended. The dam crest was raised and riprap was stockpiled. Aggregate for RCC was also processed and stockpiled during this period.

The contractor's RCC mixing plant consisted of a Barber-Green continuous mix pug mill with volumetric feed. The plant was charged from the stockpiles by front-end loaders. RCC was conveyed to the placement area by front-end loaders and spread with a John Deere 550 bulldozer. Compaction was accomplished using an Ingersoll-Rand DA-50, 10-ton dual-drum, self-propelled vibratory roller.

During the week preceding scheduled placement on the dam, an RCC test section was, constructed in order to test the mixing plant, delivery and spreading equipment, and to afford the contractor's personnel the opportunity to gain hands-on experience with RCC. In addition, the inspection staff was able to verify in-place densities achieved with differing numbers of roller passes and set performance requirements for actual RCC placement on the dam.

The contractor placed RCC 24-hours-a-day with each person working a 12-hour shift. This allowed for a smooth transition between shifts. The total contractor work force consisted of 2 supervisors, 2 plant operators, 10 equipment operators, 10 laborers, and 2 mechanics. Cement delivery was scheduled every six hours to accommodate a target production rate of 35 yd^3/hr (46 m^3/hr).

The contractor first placed an RCC access ramp under the left containment wall, working from the crest to the toe. Placement of the RCC auxiliary spillway liner started with the key-way at the downstream end of the apron. After completing the 3-ft (0.9 m) thick apron in equal 12-in (300 mm) lifts, placement proceeded up the downstream face (Figure 3). The RCC was placed in 8-ft (2.4 m) wide lanes, and each lane was offset from the previous lane by about 3-ft (0.9 m), depending on the slope. This resulted in an RCC slab approximately 3-ft (0.9 m) thick, measured perpendicular to the slope, with a stair-stepped appearance. The RCC was compacted next to the existing service spillway with power tampers.

The weather during RCC placement was cold and rainy with occasional snow flurries. Other than requiring frequent adjustment of the mixing water, the weather had no effect on operations. The pug mill performed well, with the cement feed checked periodically. The mix proportions were found to be extremely consistent. In all, about 4,800 yd^3 (3,850 m^3) of RCC were placed

Figure 3. View from downstream, looking upstream, with RCC placement about fifty percent complete. All the placement, spreading, and compacting equipment used on the project can be seen. Note the steps in the RCC liner and the left containment wall on the right side of the photograph.

continuously without mechanical breakdown in 80 hours, for an average production rate of 58 yd^3 (46 m^3) per hour.

During RCC placement, segregation and contamination were kept to a minimum. RCC wet density was tested with a nuclear density gauge and no difficulty was encountered in attaining the specified densities.

ENVIRONMENTAL IMPACT

The project is located in the Gunnison National Forest, adjacent to a campground, and the reservoir is maintained solely for recreational purposes. Therefore, preservation of natural vegetation in the area was of paramount importance. By balancing the excavation and fill volumes, and obtaining the RCC aggregate from a previously disturbed borrow area, minimal disturbance to the environment occurred. The finished project is an aesthetically pleasing structure that blends in well with the pristine surroundings (Figure 4 and 5).

Figure 4. Completed dam viewed from downstream on the right abutment.

Figure 5. Downstream view of the completed project. Note the raised crest sections at either end, the pre-existing service spillway, the extended outlet conduit (in operation), and the RCC auxiliary spillway liner.

CONCLUSION

The modifications to Spring Creek Dam were an unqualified success. By providing an auxiliary spillway using RCC, the dam can be safely overtopped by the PMF. The project was completed on time and at a substantial cost savings when compared to other conventional solutions. The work was accomplished at a remote, high mountain site with limited access and a short construction season. The finished product is a visually pleasing structure that blends in well with the natural environment.

REFERENCE

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GEOLOGICAL CONDITIONS AND TUNNELING METHODS FOR CONSTRUCTION OF THE EISENHOWER-JOHNSON MEMORIAL TUNNELS

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Key Terms: Straight Creek Tunnel, Eisenhower Memorial Tunnel, squeezing ground, continental divide, landslide, multiple drift, rock reinforcement, spiling

ABSTRACT

The history of the Eisenhower Johnson Memorial Tunnels (EJMT) in Colorado is a remarkable story of the construction of a transportation route through the Continental Divide, a process that is ongoing. Tunneling through the Continental Divide has developed in several stages, beginning during World War II and continuing through the 1960's and 1970's. In the case of the Straight Creek Tunnel (Eisenhower North Bore), construction operations came to a standstill for one year, primarily due to underestimation of the difficulties in tunneling through fault zones and squeezing ground conditions associated with the Continental Divide. All of these challenges were eventually overcome and serve as an illustration of both engineering ingenuity and the severity of tunneling challenges that exist beneath the Continental Divide in the area of Interstate 70. This paper is a summary of the underground conditions encountered while tunneling under the Continental Divide in the vicinity of I-70. The paper is based on a review of available geological and geotechnical reports, plans, and specifications as they pertain to tunnel construction of the Straight Creek (Eisenhower) and South (Johnson) tunnels. This paper was undertaken to summarize the difficulties and problems encountered during the initial tunneling operations to provide a source of information for future design considerations for proposed tunnels under the Continental Divide.

ALIGNMENT CONSIDERATIONS

A transportation route through the Continental Divide had been desired since the late 1880's to ease transportation problems associated with the early western mining camps. This natural barrier interfered with the transfer of raw materials from the western slope and supplies and tools from the eastern slope. Railroads were the first to address this problem in the 1800's and early 1900's and the first tunnel through the Continental Divide in Colorado, the Alpine Tunnel, was completed in 1893 between Buena Vista and Gunnison. Other tunnels such as the Hagerman Tunnel followed but were quite costly. The only railroad tunnel still in use that passes through the Continental Divide is the now famous Moffat Tunnel, which was completed in 1928.

Loveland Pass, which crosses over the Continental Divide, was completed in 1870 and served as a wagon road until the early 1930's. Beginning in 1931, this wagon road was made into a motor highway with a hard surface and was completed in 1950. Exploration for a tunnel to handle car traffic began in 1932 with surveys done that year and again in 1940 and 1941. Between 1941 and 1943 an exploratory tunnel bore was driven under Loveland pass for a distance of 5483 ft (1670 m). At that time, it was planned that the tunnel would be part of the strategic defense roadway network in response to World War II.

The Colorado Department of Highways, as the Colorado Department of Transportation was then known, investigated possible tunnel sites through the Continental Divide during 1943 to 1960. Several routes were considered namely Berthoud Pass, Vasquez Pass, Devil's Thumb, Jones Pass, Loveland Pass, and the Straight Creek Route. The Straight Creek site would eventually become the EMJT complex.

SITE GEOLOGY AND CONDITIONS

Topographically, the ground surface above the EJMT consists of steep mountainous terrain. The Continental Divide forms a high mountain ridge that trends northeast-southwest across the area. Currently, I-70 crosses underneath the Continental Divide in an east-west direction. West of the divide the ground is steeply sloping with gradients steeper than 1H:1V. Snow avalanche chutes are numerous in the vicinity of the west portal. Large cirques that formed as a result of glaciation formed east of the divide and are the dominant geomorphic feature in this area. Within the cirques are minor ridges, drainage systems and hummocky terrain consistent with glaciation. Surface geologic mapping was conducted by the United States Geological Survey and the results of their work were published in 1962, 1965 and summarized in 1974 (USGS, 1974). Surficial deposits consist primarily of talus, glacial moraine, and localized peat deposits. A large landslide deposit occurs above the east tunnel portals and it is likely that there are other unstable masses of highly faulted rock and fault gouge.

Bedrock at the tunnel site consists of the Silver Plume Granite and the Idaho Springs Formation. The Silver Plume Granite is a medium to coarse-grained, biotite-rich, igneous rock of Pre-Cambrian age. Biotite schist and gneiss of the older Idaho Springs Formation are metamorphosed sediments which appear as inclusions within the granite. The primary rock types within the tunnel are generally granite and meta-sedimentary gneiss and schist. The granite is gneissic, quartzitic and/or pegmatitic with biotite. It is slightly to highly weathered with the feldspar altered to clay. The foliated metamorphic rock is biotitic gneiss with schist. Pegmatite dikes crosscut the granite and meta-sedimentary rocks throughout the tunnel. The dikes range in thickness from a few inches to several feet (0.1 to 6 m). Several diorite dikes of probable Tertiary age intrude the older rocks in the west-central portion of the tunnel.

The primary structural feature within the tunnel area is a large-scale fault and fault zone feature that follows the general trend along the Continental Divide. This structural feature has been referred to as the Loveland Shear Zone, the Loveland Fault Zone, the Straight Creek Fault, the Loveland Pass Fault and the Berthoud Pass Fault Zone. In this paper the shear zone will be called the Loveland Shear Zone and the distinct fault within it will be called the Straight Creek

Fault. This fault zone varies in width and nature of displacement creating squeezing ground conditions that were encountered along the tunnel alignments.

The EJMT is located at an elevation of about 11,000 ft (3353 m) with maximum overburden cover of approximately 1,450 ft (442 m). Each tunnel is generally 48 ft wide by 50 ft (14.5 x 15.2 m) as excavated. The Continental Divide, which is the highest point, is at an elevation of 12,575 ft (3833 m). Figure 1a depicts the general layout of the EJMT complex.

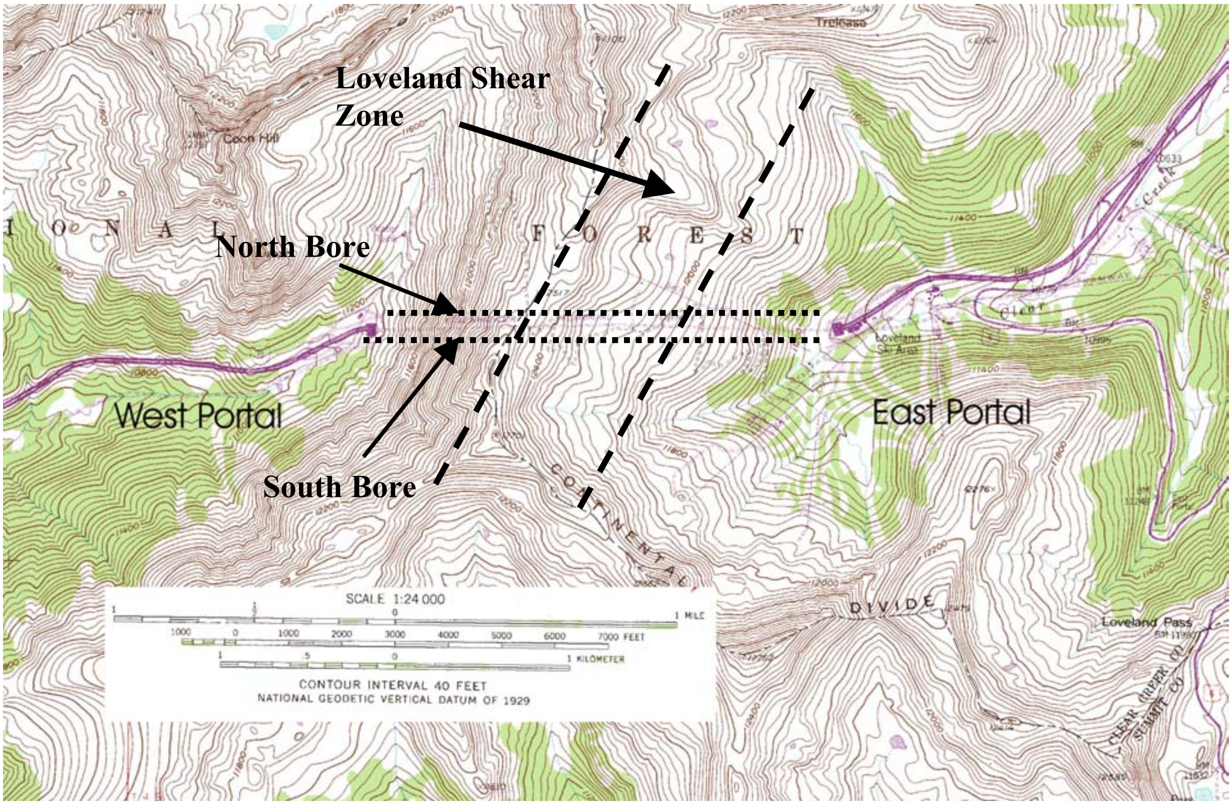


Figure 1a. General Layout of the Eisenhower-Johnson Memorial Tunnels

Another structural feature is evidence of thrust faulting approximately six miles (1.8 km) west of the west portal. This thrust fault, known as the Williams Fork Thrust, has be mapped at the surface for over 50 miles (15.3 m) in a north-south direction. There has been speculation that some of the structural features at the tunnel site have resulted from thrust forces produced by the Williams Fork Thrust.

The Loveland Shear Zone trends northeast and consists of numerous faults and shear zones of diverse orientation. Fault gouge, which is present throughout the Loveland Shear Zone, is the pulverized rock product of large scale faulting and consists of very highly altered, soft, plastic sandy clay. Triaxial tests indicated the strength of the material varies from a phi (ϕ) of 36 to 12 degrees with cohesion from 0 to 1066 lb/ft^2 (51 kPa). Other testing indicated the fault gouge material had a liquid limit (LL) ranging from 33 to 45 and a plasticity index (PI) ranging from 15 to 18. The fault gouge typically has an overburden thickness of 1,100 ft (335 m), which created most of the severe squeezing ground conditions encountered during tunneling. Squeezing ground is generally referred to as areas where the ground stress around the tunnel opening exceeds the strength of the intact or in-situ rock

Other adverse geologic conditions related to the Loveland Shear Zone included a large-scale landslide that occurred during excavation of the east tunnel portal complex. This was likely related to bedrock material on the eastern side of the Continental Divide which had been fractured by the Loveland Shear Zone creating unstable slopes prone to deep seated landslides. Based upon the history of this slope failure, it has been suggested that the ground surface within the Loveland Shear Zone can be characterized as meta-stable.

The glacial cirque overlying the eastern two-thirds of the tunnel is an area of wetlands and water intake, especially during snowmelt. The tunnels intersect deep-seated joint systems that likely transport water from the ground surface to the tunnel level. Snow melt is considerable during June and July.

TUNNELING HISTORY AND METHODS

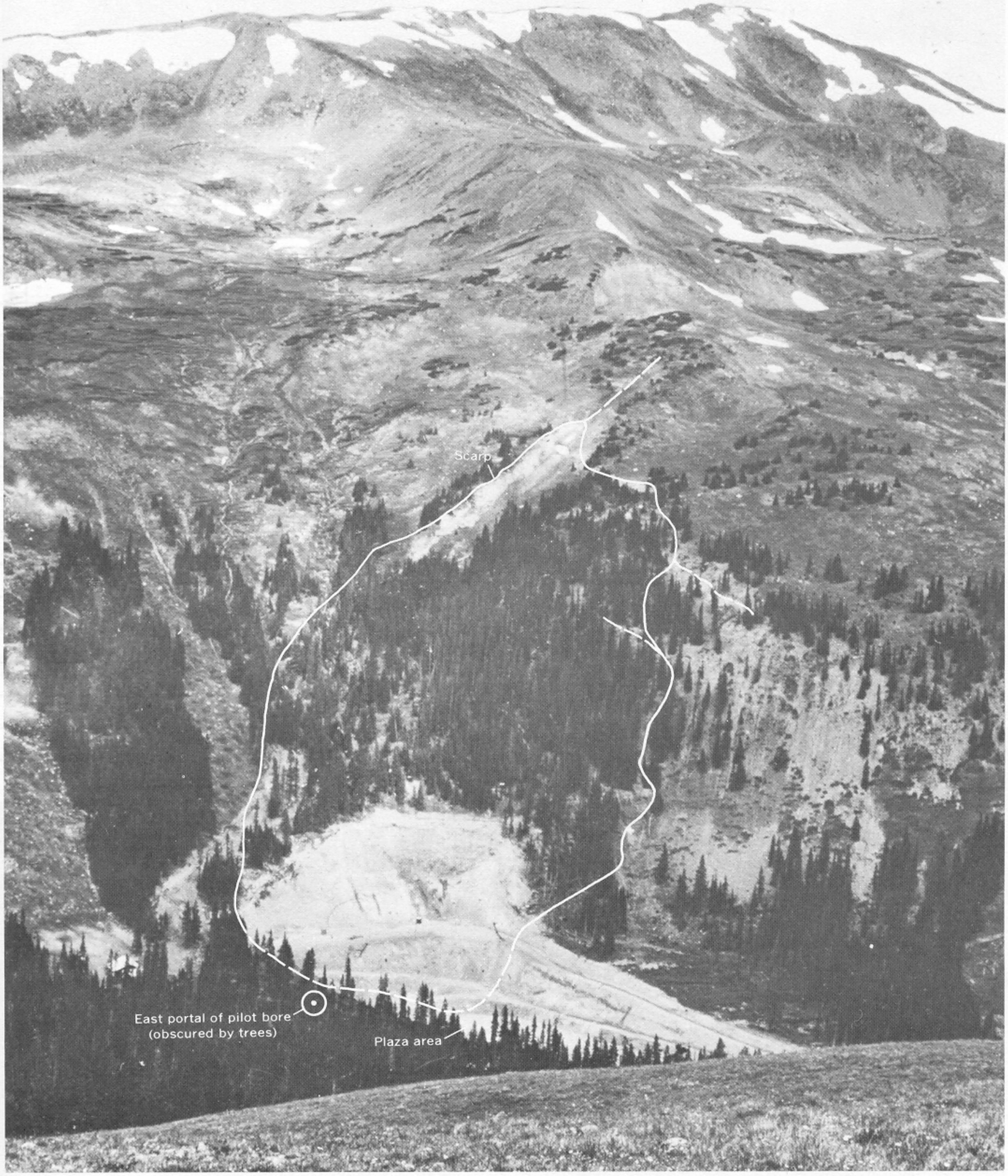
Loveland Tunnel

As discussed previously, a pioneer tunnel was constructed under Loveland Pass in 1943. The 7 ft diameter (2.13 m) tunnel was driven using drill and blast methods. The tunnel was considered to be in moderate to bad ground conditions and encountered the Loveland Shear Zone. However, based on other reviews, it appears the ground conditions in the shear zone did not exhibit the extreme squeezing nature of the fault gouge encountered in the Straight Creek area. This is likely due to the relatively shallow overburden depth of the Loveland tunnel when compared to the current I-70 tunnels. A larger tunnel diameter was not constructed at that time since it was considered cost prohibitive due to the poor ground conditions and there was an overall lack of competitive bidders at the end of WWII.

Deep-Seated Landslide During East Portal Excavation

During construction of the eastern portal of the proposed EJMT alignment, a large slope failure, or landslide, was initiated by the removal of the toe of the slope. This slope failure, which occurred in 1963, became known as the East Portal Landslide. The initial movement was estimated to encompass 2,000,000 yd^3 (1,530,000 m^3) of highly fractured bedrock. Figure 1 illustrates the landslide area in 1965. The photo is looking north.

As much as 14 ft (4.2 m) of displacement was reported at the apex of the slide in the months following activation of the failure. The fact that this was an area of historic mass movement and that the slope was in a meta-stable state was apparently not recognized until after the slope



FRONTISPIECE.—Loveland Basin landslide, view northwestward (1965).

Figure 1. Loveland Basin Landslide, 1965 (USGS, 1972).

failure. The slip plane, or slip planes, appeared to project into the tunnel but were not directly observed there. It was postulated that the movement occurred over a wide zone, or multiple planes, making the zone of displacement difficult to observe in the tunnel.

Initial analysis by the United States Geological Survey (USGS, 1972) utilizing seismic and resistivity measurements, estimated the volume of the unstable mass at between 500,000 and 770,000 yd^3 (590,000 m^3) cubic yards. The Federal Highway Administration (FHWA) analyzed the slope failure in 1964. They recommended that a 100,000 yd^3 (76,500 m^3) buttress be constructed at the toe of the slide, and that the east portal be shifted south to accommodate the buttress. A 62,000 yd^3 (47,000 m^3) buttress was constructed by September 1963. In October 1963 horizontal drains were installed and a surface structure was constructed to divert runoff and spring water from the mass. Measurements taken from October 1963 to October 1964 indicated that slope movement had decreased significantly. It was concluded that the failure mass was relatively stable at this time. Observation of the mass movement continued and in 1966 eight inclinometer wells were installed at depths between 90 and 245 ft (75 m) to monitor ground water and slope movement. Control measures undertaken in 1966 included further installation of drainpipes and diversion of surface runoff, and the in-filling of surface fractures. Movement was observed at deeper levels than those detected in 1963. Multiple slip planes were discovered extending to depths of 245 ft (75 m).

The firm Tippett, Abbett, McCarthy and Stratton (TAMS) was awarded the contract for the Eisenhower Memorial Tunnel in 1965. The contract also included monitoring and mitigating the landslide. The discovery of deeper slip planes within the slide mass indicated that the failure involved approximately 3,000,000 yd^3 (2,300,00 m^3) rather than the approximately 700,000 yd^3 (535,000 m^3) estimated initially. TAMS revised the stabilization measures based on this new information. They recommended 1,365,000 additional yd^3 (1,000,000 m^3) of compacted fill on the buttress berm and additional horizontal drains be installed along the toe of the slide. Efforts were again made to fill in the numerous surface fractures in the slide mass.

Subsequent to these efforts, slope failure continued, and movement was observed primarily in the summer months. Acceleration of movement became pronounced and concerning from 1969 to 1970. Further instrumentation involved the installation of two multiple-position borehole extensometers (MPBX's) in early 1971. These instruments provided more information on the rate of movement and location of the slip-zones. Based on this data a third and final buttress load was placed. No increase in movement was observed in the following year, in fact, monitoring of the various instruments within the slope failure through the spring of 1973 indicated a deceleration of movement. At this point the slide was considered to be stable. However, some of those working with the slide data cautioned that ongoing creep was a component of the slope failure and that the rate of this creep should be established. It is not known if any monitoring of the slide was conducted after 1974.

The original volume estimated to be necessary to stabilize the slope failure was approximately 19 percent of the volume that it finally took to stabilize the slide. The final buttress volume was 326,464 yd^3 (250,000 m^3), 5 times the 62,000 yd^3 (47,000 m^3) originally placed in 1963.

Figure 2 illustrates a generalized profile of the landslide showing multiple slip surfaces.

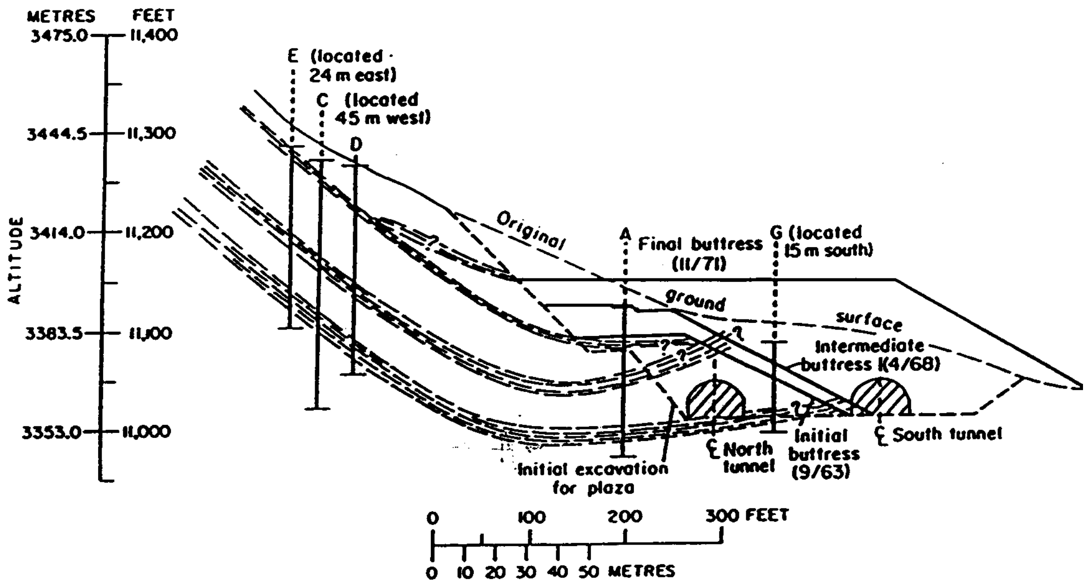


Figure 2. Generalized section through the toe of the landslide showing the approximate location of original ground surface, buttresses and major slip zones at eastern EJMT portal. (Fitzhugh, 1969).

PILOT TUNNEL

Project History

A 12 ft by 12 ft (3.6 m x 3.6 m) pilot bore was driven along the approximate alignment of the South (Johnson) Tunnel. The primary function of the pilot bore was to provide geologic and instrumentation data for construction of the two larger tunnels. The pilot bore was driven between October 9, 1963 and October 21, 1964. The initial portal location was realigned due to the constraints imposed by the landslide at the east portal. Test results obtained from the pilot bore were used in an attempt to predict more accurately the geologic conditions that would be encountered during construction of the proposed twin tunnels. Attempts were made to determine the maximum rock loads that would be encountered in the tunnels so that steel and concrete support requirements could be predicted for construction of the larger tunnels. A summary of the instrumentation results is addressed later in this paper.

The original plans for the pilot tunnel called for a timber-supported drift that was approximately 12 ft by 12 ft (3.6 m x 3.6 m). The plans called for 8 x 12 in (0.2 m x 0.3 m) posts, caps, and invert struts with 2 x 8 in (0.05 m x 0.2 m) lagging in the ribs and 3 x 8 in (0.07 m x 0.2 m) lagging in the back. Timber sets were to be installed on 3 to 4 ft (1 m) centers with a maximum of 5 ft (1.5 m) spacing. Load cells were also provided for and were to be installed every 500 ft (150 m) along the tunnel alignment.

Tunnel excavation was undertaken using drill and blast methods for a three-shift workday. It was reported that advance rates were 10 ft (3 m) per shift (30 ft/day) (9 m/day) in normal operations and 3.5 ft (1 m) per shift (10.5 ft/day) (3.2 m/day) in the poor rock zones. Two months after starting the work, the contractor requested the use of steel sets instead of the timber sets. Two piece 4-in (0.1 m) I 7.7 lb steel sets were approved. The steel sets had an outside rib radius of 14.3 ft (4.3 m) and an outside back radius of 5.5 ft (1.6 m). When the tunneling operations encountered poor rock conditions the 4 inch steel sets began to yield and deform requiring installation of 6-in (0.15 m) H 25 lb steel sets with a radius of 14 ft (4.2 m) and outside back radius of 5.16 ft (1.57). Highly altered fault zone sections were encountered requiring 6-in (0.15 m) steel sets on a 2 to 4 ft (1 m) spacing. Reportedly the floor heaved approximately 2 ft (0.6 m) in a two month period in one section. Loads on the instrumented 6-in (0.15 m) steel sets were reported to range from 25 kips to 225 kips (100 to 1000 kN).

It was reported that the groundwater flow, which was as high as 150 gpm would generally taper off over the course of 10 days to no more than a trickle.

NORTH (STRAIGHT CREEK - EISENHOWER) TUNNEL

Historical Overview

Tippetts-Abbett-McCarthy-Stratton (TAMS) provided the design plans for construction bids on October 3, 1967. Included with these plans were all the geologic data and engineering reports from the pilot bore. The bid for the construction contract was awarded to Straight Creek Constructors, a joint venture of Gibbons & Reed, Western Paving, Al Johnson Construction and Kemper Construction Company.

Tunneling began on the west side of the Continental Divide on March 11, 1968. A top heading was advanced for approximately 4,300 ft (1310 m) to the east. Excavation of the bench proceeded for approximately 1,600 ft (490 m) to the east, when a fracture zone was encountered and approximately 60 ft (18 m) of the tunnel collapsed. It was later determined that clay filled joints in a fracture zone contributed to the collapse. Despite this setback most of the top heading and bench areas were excavated to approximately Sta. 81+57 (see Figure 3) by July 1969. At this station, the tunnel alignment reached the most severely faulted section of the Loveland Shear Zone. This section consisted of an approximately 160-ft (48.8 m) section of squeezing fault gouge. The materials had the characteristics and consistency of sandy stiff clay. The contractor attempted to use a tunnel shield while advancing through the squeezing fault gouge. The shield was advanced approximately 70 ft (21 m) through the fault gouge when, after roller and skid system problems, it became permanently stuck.

Excavation from the east portal began in early 1968 using the top heading and bench method. Numerous overbreaks and fallouts occurred due to a thin cover of highly fractured rock overlain by the fill of the buttress berm. By late 1969 the top heading had been completed approximately 2800 ft (854 m) to the west and the bench had been excavated 500 ft (152 m) to the west. As the top heading penetrated further west into the Loveland Shear Zone, and the overburden thickness became greater, many of the steel sets began to deform and fail as large loads began to develop.

The failure of the top heading steel sets was accompanied by extreme side-wall convergence and floor heave at various locations. Substantial additional cost was incurred and the contractor filed a change of condition lawsuit against the Department of Highways. Reappraisal of this section and the western sections of the tunnel caused the headings to be shut down for approximately one year.

Mathews Engineering, Inc. took over the construction in December 1970. The tunneling method was modified to strengthen the existing steel sets in the eastern end of the tunnel. For the western end of the tunnel that encountered the squeezing fault gouge, it was also decided to use a multiple drift method to support the perimeter around the proposed tunnel profile with 8 ft by 8 ft (2.5 m x 2.5 m) drifts that were filled with concrete. In the fault gouge zone the multiple drift excavation began from the west on December 14, 1970 and from the east on January 4, 1971. The tunnel was successfully excavated through the most severe section of the Loveland Shear Zone on July 24, 1972. By October 1972 the convergence within most of the zone had been stabilized. The tunnel was opened for traffic on March 8, 1973.

Tunneling Method

The original tunnel plans consisted of top heading and bench excavation using steel support sections that ranged from 12WF106 arch ribs in the better ground conditions to 14WF287 invert struts in the poorer ground conditions. Reportedly the steel set spacing in the poorer ground areas ranged from 2 to 4 ft (0.6 to 1.2 m).

By the end of 1970 the top heading and bench tunneling method was modified to account for the difficult geological conditions present in the tunnel. The tunnel alignment was divided into five zones based on what had been excavated and the ground conditions anticipated. Detailed mapping was done and included data on the amount and type of support, load data, and information on replaced supports, fallouts and overbreak. Figure 3 illustrates a generalized profile of the tunnel alignment with the 5 zones.

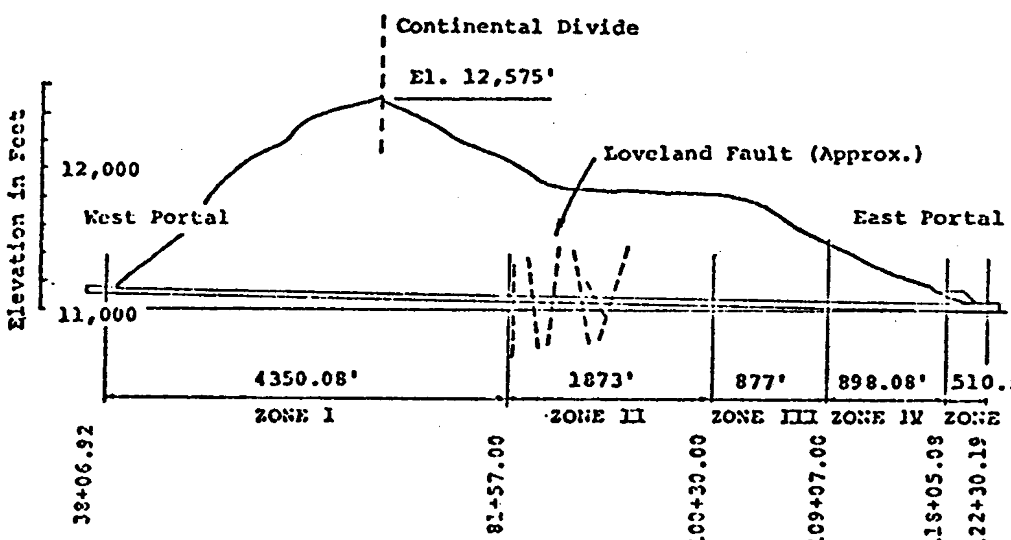


Figure 3. Generalized profile of the Straight Creek (Eisenhower) Tunnel (Leeds, 1974).

Figure 4 depicts a generalized elevation, plan and section view of the modifications to the top heading and bench tunneling method. The figure shows the unshaded sections of tunnel as excavated and the shaded areas as unexcavated. As illustrated in the figure, the Straight Creek (Eisenhower) Tunnel runs approximately east to west and is approximately 8,900 ft (2,700 m) long.

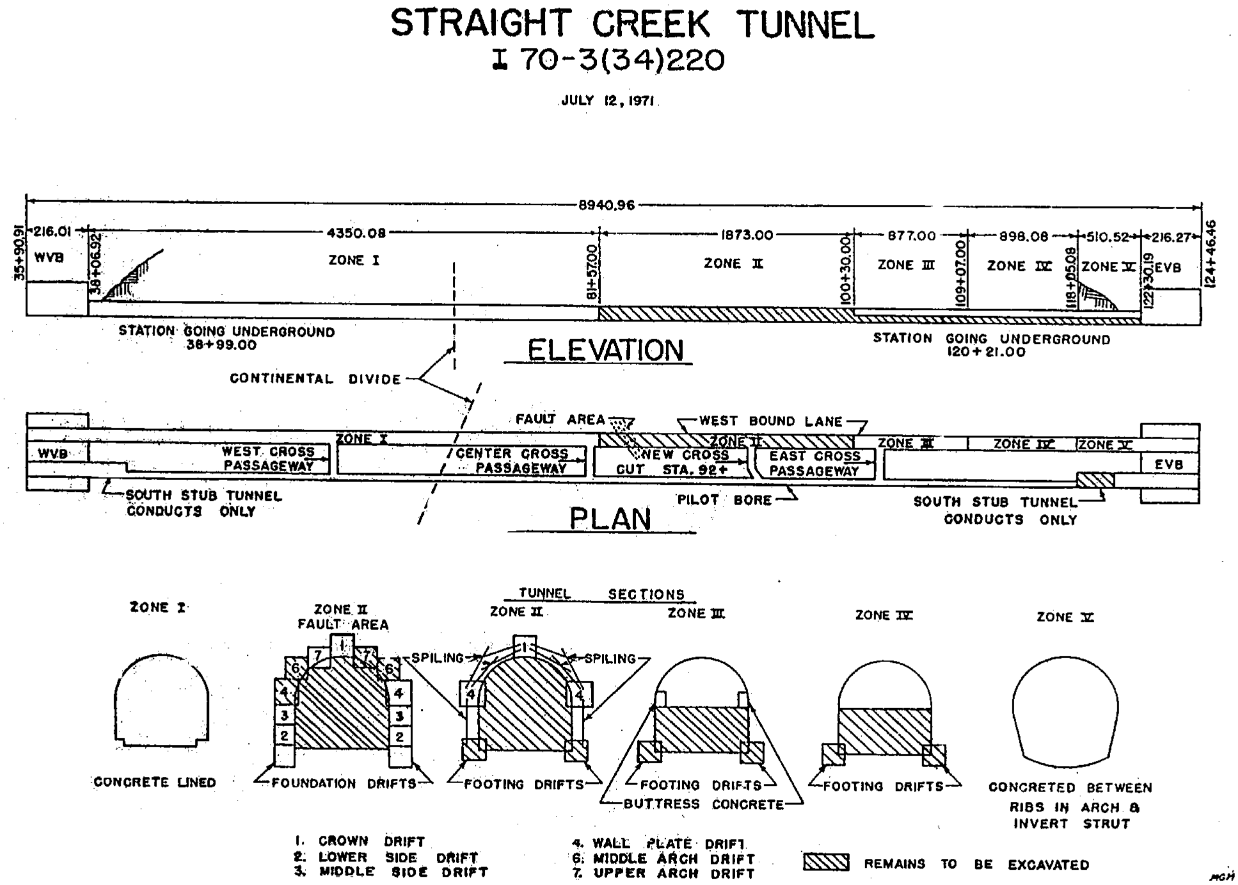


Figure 4. Elevation plans and profile for tunneling of the Straight Creek Tunnel.

Zone II, which is 1,873 ft (571 m) long, was considered to have the worst ground conditions with an approximate 160 ft (50 m) section of squeezing fault gouge. Zones III and IV were also considered to within the Loveland Shear Zone, but the ground conditions were not as difficult as Zone II and typically have less overburden thickness. Before bench excavation of the eastern end of the tunnel could be completed it was necessary to stabilize the top heading of Zone III, IV, and V. Zone III was reported to have exhibited the most severe deformation of the steel arch sets. Steel sets in this zone were excessively deformed as a result of ground movement. It was necessary to completely encase the steel ribs in shotcrete and concrete. To further strengthen the support system, a 5 ft wide by 6 ft high (1.5 m by 1.8 m) concrete buttress was installed at the wall plate or base of the upper arch sets. The buttress was designed to increase the bearing capacity of the arch system. Additional No. 8 reinforcing grouted rock bolts, 20 ft (6 m) in length and on a 5 ft by 5 ft (1.5 m by 1.5 m) spacing were installed over the entire archway to reduce deflection of the archway. All of these measures appear to have been successful in limiting deflection of the upper archway system.

For bench excavation on the eastern end of the tunnel in Zones III, IV, and V, it was necessary to install grouted No. 14 reinforcing rock bolts, 20 ft (6 m) in length laterally outward from the tunnel. Spiling was also installed down the side walls and upward from the footing drifts to provide lateral resistance. Spiling consisted of No. 11 and 14 reinforcing bar that was grouted in various lengths, typically 20 ft (6 m). Figure 5 depicts the reinforcement system for these sections.

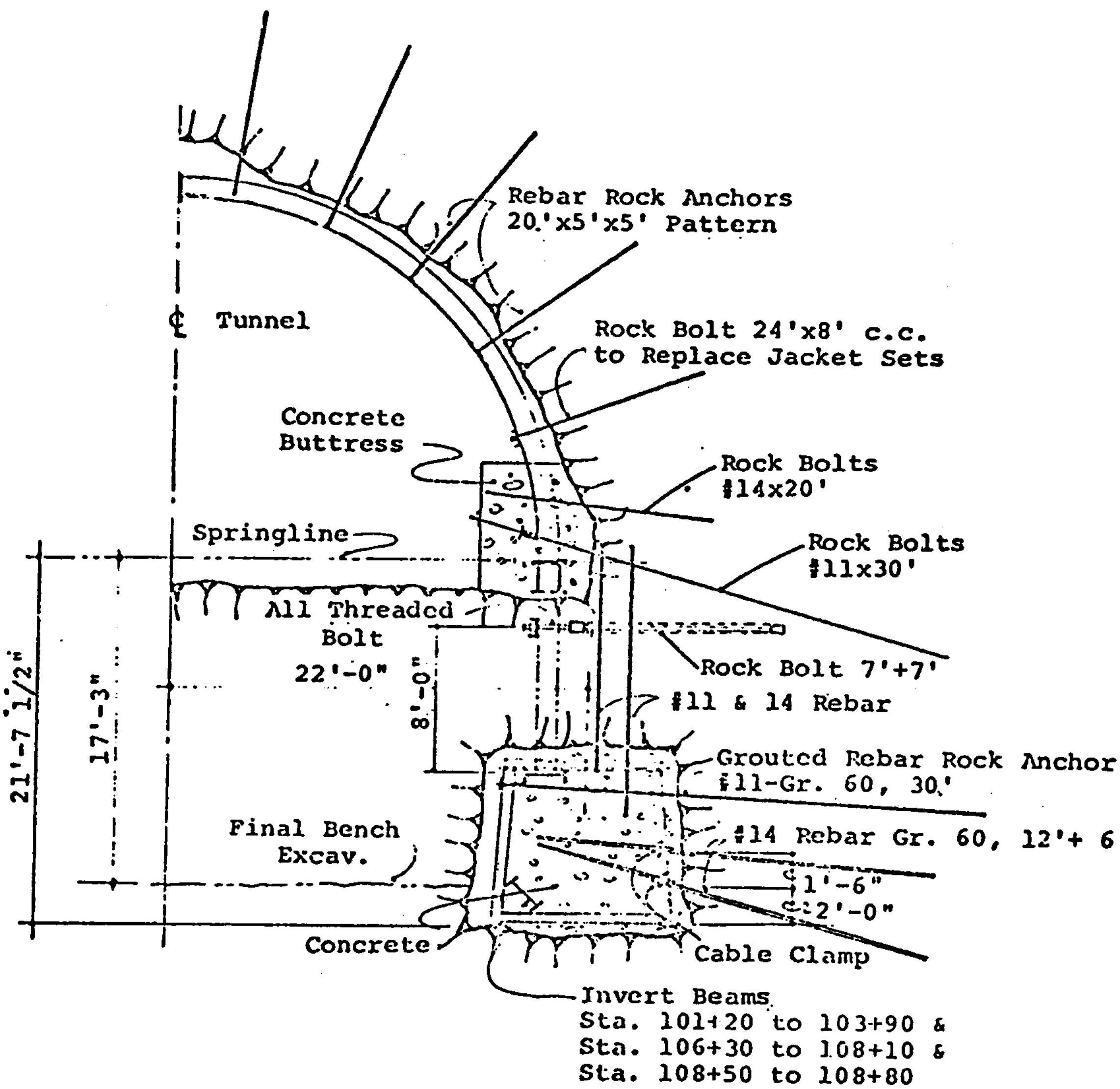


Figure 5. Zone III Rock Reinforcement and Support Sta. 100+30 to Sta. 109+07.

The tunnel section along Zone II exhibited the worst squeezing ground conditions. At these points, the ground will tend to deform plastically and squeeze into the tunnel opening. To tunnel through Zone II and the approximate 160 ft (50 m) squeezing fault gouge it was proposed to construct the multiple concrete filled 8 ft by 8 ft (2.4 m by 2.4 m) drifts to support the perimeter around this section of the tunnel. For the majority of the tunnel through Zone II, only five drifts had to be excavated around the perimeter. These consisted of a crown drift, two wall plate drifts, and two footing drifts. Figure 6 depicts the generalized section. The crown drift was completely filled with concrete and acted as a top beam. The wall plate drifts were situated so they were located at the steel rib side post / wall plate location (near springline) for the steel sets. Sections of the side post of the steel set were placed and blocked out in the wall plate drift. Concrete was then filled in the wall plate drift between the steel rib and wall. A concrete footing was poured in the footing drifts, however they were not filled with concrete. Spiling in the form of sections of re-bar were drilled and grouted upward and downward from the wall plate drifts and downward from the crown drift to provide additional lateral tunnel support. After completion of the drifts the top heading was driven using a breast board jumbo supported on rails.

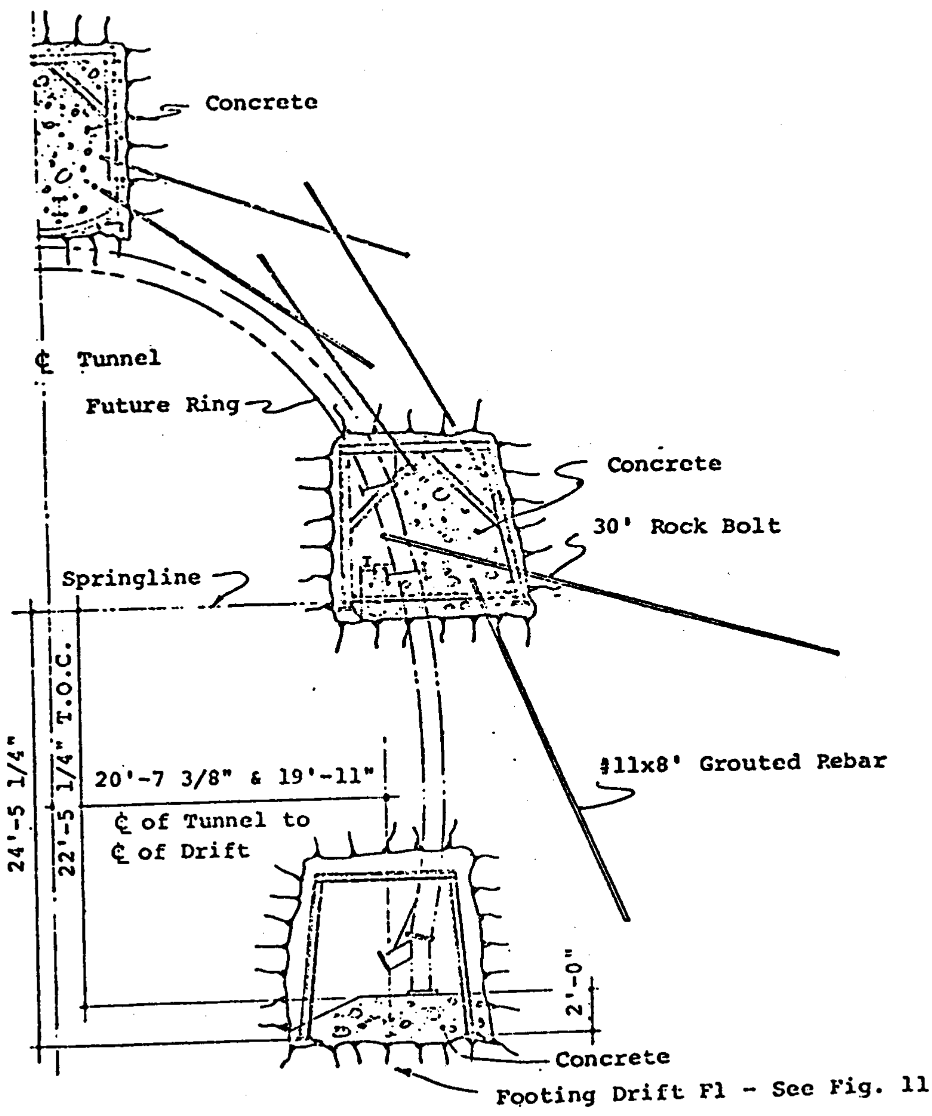


Figure 6. Zone II – Five Drifts and Rock Reinforcement Sta. 85+60 to 100+30.

In the main fault gouge section from Sta. 82+40 to 84+00 it was necessary to use 13 drifts around the perimeter of the tunnel section. Figure 7 depicts the location of the 13 drifts. As illustrated, some of the drifts were completely filled with concrete while others were blocked and poured with the steel arch ribs so the arch sets could be completed later. It was necessary to provide more bearing capacity in this section therefore the footing drifts were filled.

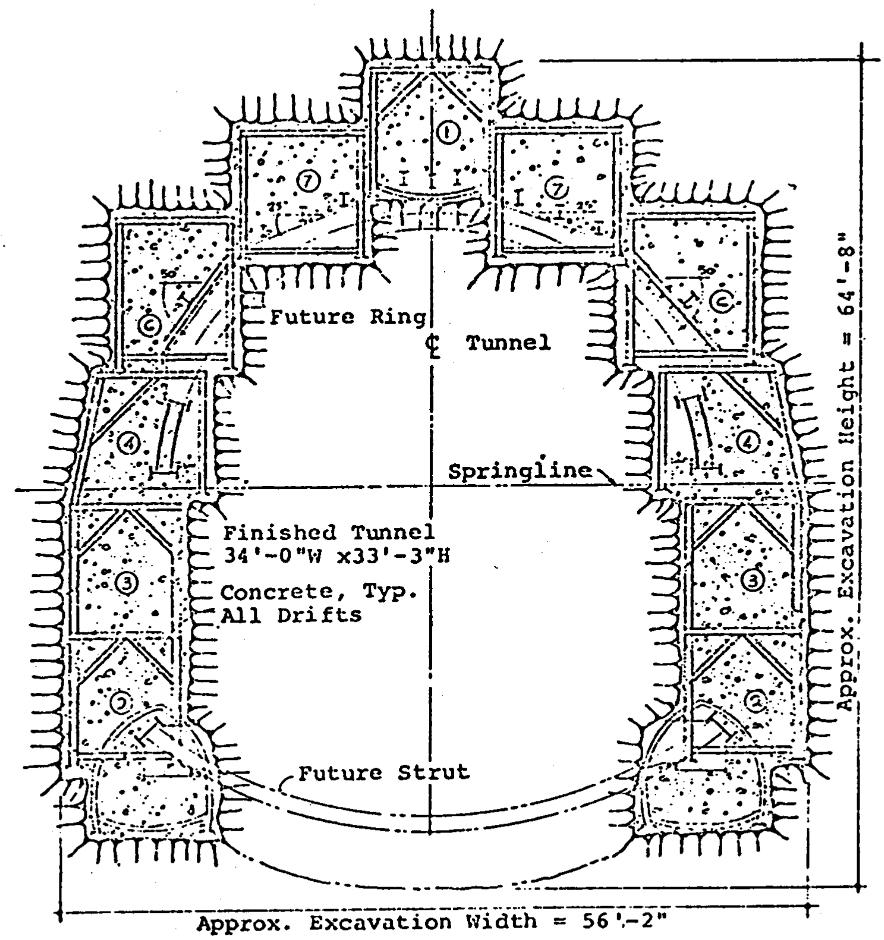


Figure 7. Zone II – Multiple Drifts Sta. 82+40 to 84+00.

It was reported that the multiple drift method was designed to provide a certain amount of deflection of the ground materials before placing the stabilizing support system. Typically during construction of the crown drifts deflections of up to 0.5 in (12 mm) could be seen in the face within 1 hour of mucking. Within one week ground movements of up to six to twelve in (15 to 30 cm) were reported in the wall ribs. Timber that was placed typically crushed and was replaced two or even three times.

Excavation of the top heading between Sta. 87+00 and 89+00 was done in many cases with a backhoe. In general, if the material wasn't excavated with a backhoe, it tended to break easily requiring light blasting loads. It was reported that the transverse spiling between the crown and wall plate drifts performed well in controlling overbreak, fallout, and raveling. The ground broke through the inner row of spiling but did not break through the outer row. The following is a generalized summary of the zones.

- Zone I - Sta. 38+99 to 81+57 (4,258 ft) (1300 m) - Reportedly competent granite. A few sections were heavily fractured causing a 60 ft (18 m) section of the tunnel to collapse during benching operations.
- Zone II - Sta. 81+57 to 100+30 (1,873 ft) (571 m) - Reportedly the worst ground conditions encountered during tunneling construction. At least 160 ft (50 m) of this section consisted of squeezing fault gouge that required multiple drifts around the perimeter of the tunnel section. The rest of the section required up to five drifts around the boundary of the tunnel section. Spiling was used to reinforce the tunnel excavation. Multiple faults and fault zones encountered.
- Zone III - Sta. 100+30 to 109+07 (877 ft) (270 m)- Reportedly a bad ground section that required additional concrete buttress support for bearing capacity of the steel sets due to high loading. Footing drifts were also required. Multiple fault zones but not as severely faulted as Zone II.
- Zone IV - Sta. 109+07 to 118+05 (898 ft) (270 m)- Tunnel excavation encountered fault and fracture zones but overburden in this area was generally less than 500 ft (150 m).
- Zone V - Sta. 118+05 to 120+21 (216 ft) (65 m)- Shallow section of the tunnel. Alignment is under the east portal landslide.

Figure 8 illustrates a conceptual model that was developed to show the various tunneling methods and stages.

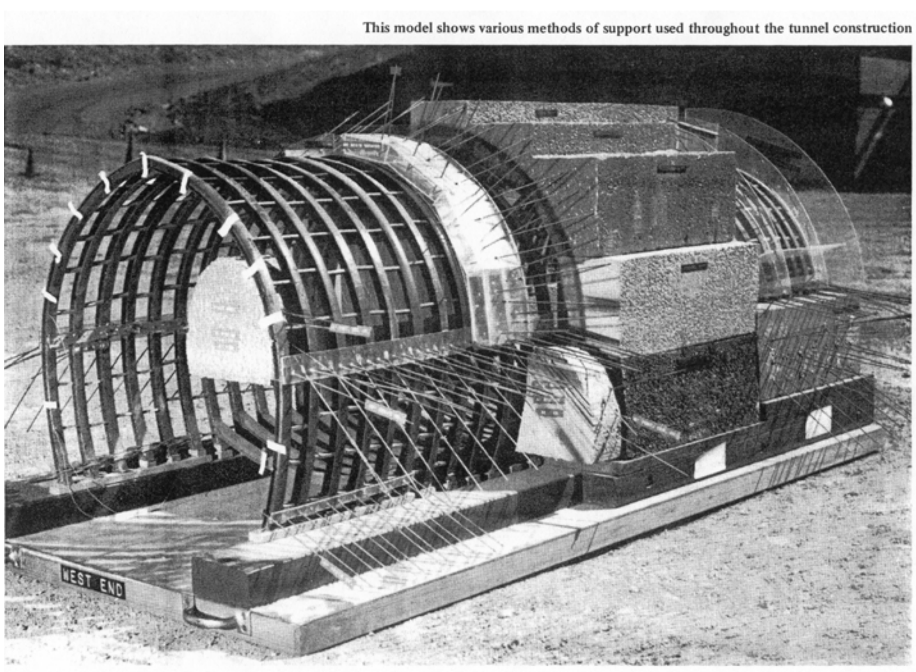


Figure 8. Conceptual model of tunneling method for I-70 bores through the Continental Divide.

SOUTH (JOHNSON) TUNNEL

Lessons learned on the Straight Creek (Eisenhower) Tunnel construction were used to construct the South Tunnel Bore. Construction began in August of 1975 with final completion and opening on December 23, 1979. This tunnel was also driven with a top heading and bench method with the use of multiple drifts where poor rock or squeezing ground conditions were encountered. Spiling was also used extensively for tunnel support.

According to a 55-minute video presentation regarding the construction of the South (Johnson) Tunnel, the steel support was designed to resist an anticipated 5,000 to 30,000 lb/ft^2 (240 to 1,400 kPa) vertical rock loading and a 0 to 18,000 lb/ft^2 (0 to 860 kPa) horizontal rock loading in the western half of the tunnel. The top heading was drilled with a rail-mounted jumbo that consisted of 12 drills mounted on two levels. The jumbo was also used to drill spiling (forepole) umbrellas ahead of the working face. The spiling appeared to consist of # 11 re-bar that was resin bonded. The lengths appeared to be between 20 and 30 feet (6 to 9 m). The spiling was placed 30 degrees above horizontal. The video reported that the spiling or pre-reinforcement bars were instrumented to indicate tension and bending in the bar with eight strain gauges in opposing pairs of selected bars. The results of the instrumentation indicated that the bars experienced up to 30 tons (60 kips) (270 kN) of tension but with relatively little bending. It was also reported that stress increases in the spiling bars were up to 1000 lb/in^2/hr (50 kPa/hr) during excavation operations. Steel sets were reported to consist of 14 in (35.5 mm) wide flange steel, on a 4 ft (1.21 m) spacing that were placed in the top heading. Two stages of concrete lining were placed, an initial concrete lining that was used to encase the steel sets and a final lining, which consisted of an internal drainage system to prevent groundwater from entering the tunnel excavation.

The video indicated that the crown drift and two side wall drifts were used on much of the east side of the tunnel. At least two, 12 ft, # 11 re-bar rock reinforcements were placed on each side of the crown drift on a 4 ft (1.21 m) spacing. At least three, 18 ft (5.5 m), # 11 re-bar rock reinforcements were placed in the side of the sidewall drifts, on a 4 ft (1.21 m) spacing. The steel set design loads for approximately 3266 ft (1000 m) of the east side tunnel headings ranged from a 12,000 to 30,000 lb/ft^2 (575 to 1435 kPa) vertical load, and a 5,000 to 12,000 lb/ft^2 (240 to 575 kPa) horizontal load. The east heading of the tunnel was in a highly fractured rock. The video also indicated that the full multiple drift tunneling method was used for approximately 500 feet (150 M) of the tunnel in the vicinity of the Straight Creek Fault. Steel sets were not used through this section since the concrete filled multiple drifts appeared to have provided adequate excavation support, however, steel invert struts were still placed to withstand the high horizontal loads that were anticipated. It was reported the expected rock loads for this section of tunnel were a 50,000 lb/ft^2 (2,400 kPa) vertical load and a 38,000 lb/ft^2 (1,800 kPa) horizontal load.

The video reported that based on instrumentation results, the spiling support resisted approximately 25 percent of the loading conditions anticipated. The video also indicated that the loads on the steel sets were approximately 15 percent of the overall loading conditions anticipated. In general the spiling and rock reinforcement were the primary stabilizing factors in the tunnel excavation and the lining was a secondary support measure.

Quantities reported in the video were 500,000 yd^3 (380,000 m^3) of rock removed, 220,000 yd^3 (168,000 m^3) of concrete placed, 708,0000 linear ft (215,000 m) of rock reinforcement, 2,000 tons (1,785 tonnes) of reinforcing steel, and 18,000 tons (16,000 tonnes) of structural steel used in construction.

INSTRUMENTATION

During construction of the Pilot Bore, Straight Creek (Eisenhower) Tunnel, and the South (Johnson) Tunnel various methods were used to instrument the loads on steel sets and in-situ materials. The instruments placed included load cells, borehole extensometers, bar extensometers, and Gloetzl cells. After 1970 the company of Leeds, Hill and Jewett, Inc. directed the instrumentation program and added the use of electrical strain gauges to measure the loads on the steel sets. They continued to use the Gloetzl cells to measure the contact pressure between the concrete lining and bedrock and to correlate with the strain gauge data.

Straight Creek and Pilot Bore Instrumentation

The South Dakota School of Mines conducted an in-situ rock test in a cross cut between the Pilot Bore and Straight Creek Tunnel using a borehole strain rosette relief technique to determine the vertical stress of unaltered rock. The vertical stress (σ_v) was reported to be approximately 130,000 lb/ft^2 (6,225 kPa), the north-south horizontal stress (σ_N) was approximately 58,000 lb/ft^2 (2,700 kPa), and the east-west horizontal stress (σ_E) was approximately 14,500 lb/ft^2 (700 kPa). The calculated vertical stress would be (σv_{cal}) 161,000 lb/ft^2 (7,700 kPa), assuming an overburden depth of 1,000 ft (300 m) and a rock density of 161 lb/ft^3 (25 kN/m^3). A flat jack technique was used to measure the in-situ rock stress near a faulted section of the Pilot tunnel at Station 85+00 which indicated an in-situ stress of 6,000 to 9,000 lb/ft^2 (290 to 430 kPa). The flat jack test appears to have been performed a number of years after substantial completion of the Pilot Tunnel.

Load cells were also placed behind multiple steel sets in the Pilot Bore. Reportedly for the 4 and 6 in (10 to 15 cm) steel set systems used, the measured steel set rock load ranged from 200 to 6300 lb/ft^2 (10 to 300 kPa) between stations 59+35 to 118+24. The highest loads occurring at Sta. 90+50.

A total of eight steel sets were instrumented in the Straight Creek Tunnel from Sta. 73+01 to 100+67 at spacings of 4 ft and 2 ft. The instrumentation indicated that rock loads ranged from 700 to 2,600 lb/ft^2 (33 to 125 kPa). In either case, the report does not indicate how long between initial excavation and placement of load cells and steel sets or how much deflection of the ground occurred prior to steel set placement.

South (Johnson) Tunnel

A total of 45 steel ribs in the top heading and bench sections were instrumented in the second bore of Johnson Tunnel. Strains in the steel ribs and lateral convergence of the steel ribs at the springline were measured. Strain instrumentation results indicated for heavy steel ribs spaced on

4-ft (1.22 m) centers in the poorer quality ground the pressure on the steel sets ranged from 7,000 to 14,000 lb/ft^2 (335 to 670 kPa). The results also indicated that for steel ribs in good quality ground, the pressure on the steel sets ranged from 1,000 to 3,000 lb/ft^2 (50 to 143 kPa). Rib convergence was reported to be 1 in (2.5 cm) or less in most sections.

Instrumentation of the multiple drift section of the tunnel was conducted between Sta. 82+53 to 87+56. The results of the instrumentation indicated that lateral convergence of the springline varied between 0.65 and 2.3 inches. Lateral convergence at the invert level ranged from 2 to 5 inches. Stress cells on the exterior of the concreted drifts indicated pressures ranging from 14 to 72 ksf. It should be noted that pressure measurements on the sides of the multiple drifts appear to have been taken after initial deflection of the ground had occurred within the drifts. It was also reported that heave of the floor of the top heading prior to excavation of the bench ranged from 1.4 to 15.5 in (3.5 to 38 cm).

SUMMARY

This report is based on a review of available geological and geotechnical reports, plans, and specifications as they pertained to tunnel construction of the Straight Creek (Eisenhower) Tunnel, South (Johnson) Tunnel, and the Pilot Tunnel. The information given in this report was primarily based on research and review of documents that are kept by the Colorado Department of Transportation at the EJMT complex. Proposed tunnel alignments north or south of the existing tunnels may encounter substantial variation in the nature and extent of subsurface conditions in the area necessitating the need for a more comprehensive geologic/geotechnical investigation for more detailed evaluation of future tunnel alignments.

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CONDITIONS ENCOUNTERED IN THE CONSTRUCTION OF STANLEY CANYON TUNNEL COLORADO SPRINGS, COLORADO

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Key Terms: tunnel, TBM, granite, shear zone, ground water inflows, boreability, laboratory testing

ABSTRACT

This paper documents the construction of the Stanley Canyon Tunnel, the geologic conditions encountered, boreability of the rock, and the impacts of the conditions encountered on the construction. The 3.1-mi long (5.05 km), 11.4-ft (3.47 m) diameter tunnel was driven through granites with a refurbished tunnel boring machine (TBM). Difficult ground conditions encountered included shear zones, high ground water flows, and hard, tough, and strong rock. Detailed geotechnical studies during construction revealed that microscopic changes in the rock grain boundaries due to contact metamorphism resulted in the hard boreability. Originally scheduled for five months, the tunnel construction required nearly three years, including periods of project shutdown.

INTRODUCTION

The Stanley Canyon Tunnel is part of a water transmission project for the City of Colorado Springs, Colorado. Located along the Front Range of the Colorado Rocky Mountains, the tunnel delivers raw water from Rampart Reservoir to a new water treatment plant at the mouth of the tunnel northwest of Colorado Springs adjacent to the U.S. Air Force Academy. The reservoir is located in mountainous terrain at an elevation of approximately 8900 ft (2713 m). Water from the reservoir enters a drop shaft approximately 1,500 ft (457 m) deep which intersects the tunnel. The tunnel slopes at three percent for a distance of 16,582 ft (5,054 m) to the portal at the water treatment plant, which is located at the base of the mountain front. Planning is underway for construction of a future hydroelectric plant.

General Geology

The project area is near the transition between the sedimentary bedrock strata of the foothills and the igneous rocks, which form the core of the Front Range. A major frontal fault, the Rampart Range Fault, trends north-south near the portal of the tunnel. East of the fault the bedrock consists of sedimentary strata that dip eastward, away from the mountain front. West of the fault the bedrock consists mainly of granite. Figure 1 shows the general geology in the area of the Stanley Canyon Tunnel. There are two granite units that have been mapped along the tunnel alignment. The main body of granite is the Pikes Peak Granite. The Pikes Peak Granite consists of a medium- to coarse-grained pink granite with irregularly shaped mafic minerals and large feldspar crystals. Locally, a younger granite, the Windy Point Granite, has intruded the Pikes Peak Granite. The Windy Point Granite is a fine- to medium-grained pink granite with regular "leopard spot" clots of mafic minerals, mainly biotite. The Windy Point Granite forms irregular stocks within the Pikes Peak Granite. A forked stock of Windy Point Granite has been mapped intersecting the tunnel alignment approximately mid-way along the tunnel.

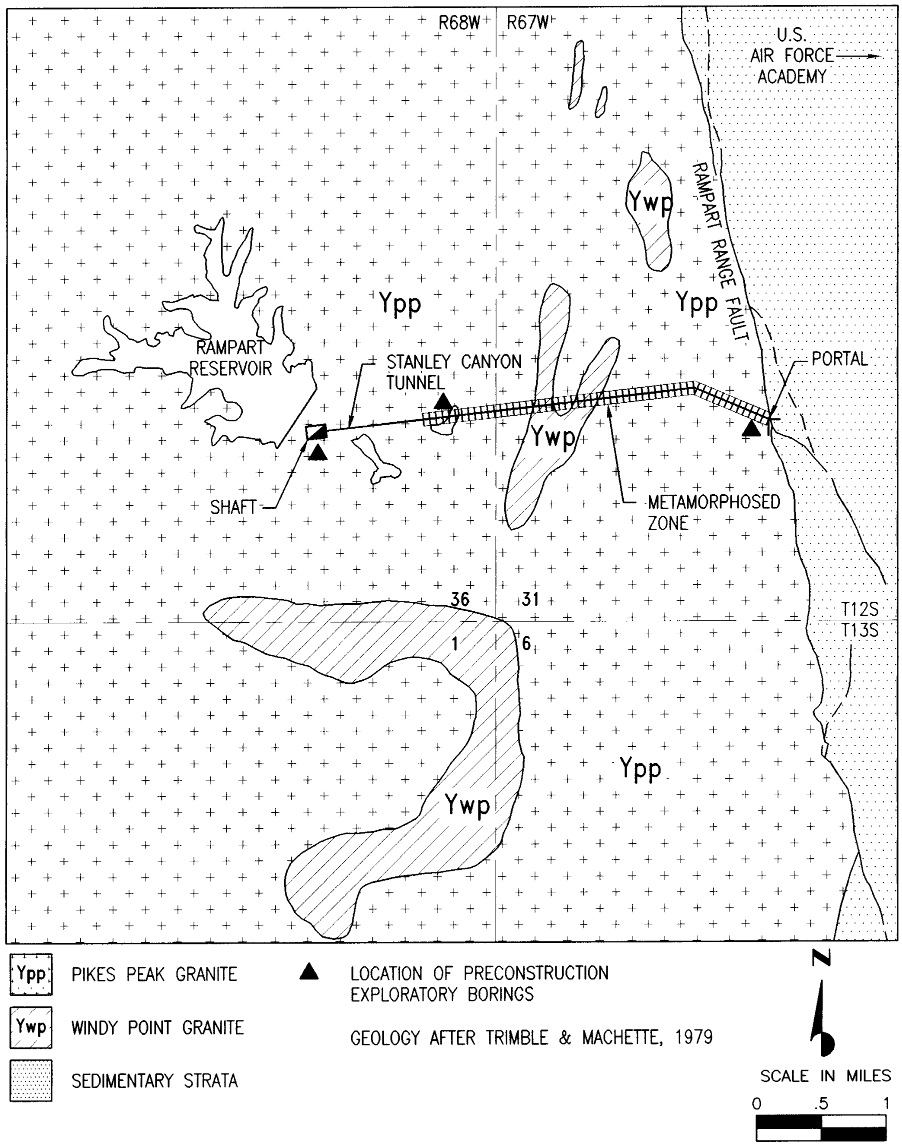

Figure 1. General Geology

Pre-Construction Geotechnical Investigations

The geotechnical investigations for the project included drilling exploratory core borings, primarily at the portal and the shaft at each end of the tunnel. The one boring that was drilled along the alignment was located to investigate a shear zone. This boring was terminated well above the tunnel elevation. In addition to the exploratory borings, the geology at the ground surface was mapped along the tunnel alignment.

Based on the geotechnical investigations, approximately 91 percent of the tunnel was expected to be excavated in Pikes Peak Granite. The remaining nine percent of the tunnel was expected to be excavated in the Windy Point Granite.

Intact Rock Properties

Laboratory testing was performed to characterize the intact rock properties of the Pikes Peak Granite and the Windy Point Granite. The majority of the rock that was tested came from borings drilled at the shaft. Based on the test results, the Pikes Peak Granite was interpreted to be a medium strength rock. The intact samples that were tested for unconfined compressive strength gave test values ranging from 6,120 psi (42 MPa) to 21,200 psi (146 MPa), averaging approximately 14,651 psi (102 MPa). The unconfined compressive strength of the granite was interpreted to range from 12,500 psi (66 MPa) to 21,000 psi (144 MPa). These strengths were similar to strengths reported for the Pikes Peak Granite at the North American Air Defense (NORAD) Facility south of Colorado Springs. Petrographic analysis of samples of the Pikes Peak Granite indicated that with the exception of biotite, the mineral boundaries in the granite were cuspate to mutually curved boundaries. This was described as simple locking of contacts, which allow the easiest breakage along grain boundaries with the least amount of energy consumption. The laboratory testing indicated that the Windy Point Granite was a high strength rock. Unconfined compressive strength tests gave test values ranging from 16,000 psi (110 MPa) to 31,000 psi (213 MPa). Mid-range of the samples tested was approximately 23,500 psi (162 MPa). Petrographic analysis of samples of the Windy Point Granite indicated more complex grain boundaries than the Pikes Peak Granite with a high degree of interlocking. This indicates more energy would be required for breakage along grain boundaries in the Windy Point Granite.

Anticipated Rock Tunnel Conditions

Based on the geotechnical investigations, the geologic setting, and the geologic mapping, it was concluded that the rock tunnel conditions at the Stanley Canyon Tunnel favored excavation with a full-face TBM. Because most of the tunnel was to be excavated in the medium strength Pikes Peak Granite, the rock was expected to have good boreability properties allowing instantaneous penetration rates (IPR's) in the range of 12 to 15 ft per hour (3.6 to 4.5 m per hour). Although shear zones were anticipated, the shear zones were expected to be relatively narrow, on the order of 10 ft (3 m) in maximum width, and extremely adverse ground conditions were not expected. Total ground water inflows into tunnel were expected to be less than 1,000 gpm (63 liters per second). Flush flows from a single zone were not expected to exceed 250 gpm (16 liters per second) from any single zone. The geotechnical investigations indicated that the tunnel conditions for the construction of the Stanley Canyon Tunnel should be generally favorable.

TUNNEL CONSTRUCTION

Morrison-Knudsen was awarded the contract for the construction of the Stanley Canyon Tunnel in the fall of 1988. The contractor chose to drive the tunnel with a Robbin's Full-Face TBM with a domed cutter head and 15.5 in (39.37 cm) constant cross-section cutters (Model No. 1210-18S manufactured in 1977). The machine was designed primarily for hard competent rock, but had a finger shield to help prevent caving in zones of blocky ground. The TBM had excavated hard rock tunnels in Europe and was reconditioned for the Stanley Canyon Project. Mucking was to be handled with a continuous overhead conveyor. The excavation of the tunnel was scheduled for approximately five months.

Tunnel excavation began in January of 1989. From the portal to Station 00+83 the tunnel was excavated in Pikes Peak Granite using conventional drill and blast techniques. T he TBM was launched at Station 00+84 on January 24, 1989.

On March 30, 1989 the TBM advance was stopped at Station 07+85 by a major shear zone. Rock within the zone exhibited fast raveling to cohesive running ground (soil-like behavior) with very short stand-up times. The granite was mechanically crushed to sand and gravel sized material. The TBM was freed and removed from the tunnel on May 20, 1989. Stabilization of the tunnel heading was completed by June 2, 1989, and a probe hole was cored ahead to evaluate ground conditions. The probe hole indicated very blocky rock conditions with local shears for the next 80 ft (24 m) and significant water pressure.

From approximately Station 08+00 to Station 10+85 the tunnel was advanced using conventional drill and blast techniques. The ground was supported by horseshoe steel sets. As the conventional tunneling proceeded, the owner and the contractor considered various options for continuing the work. The main options considered included:

- Re-launching the existing TBM with some minor repairs and modifications.
- Ordering a new, fully shielded TBM designed to handle ground conditions ranging from hard rock to soft and raveling ground.

It was decided to continue the work with the original machine. The TBM was launched from Station 10+85 in the spring of 1990 after making some modifications to the machine. On June 28, 1990 at approximately Station 21+35 the TBM encountered another major shear zone. The rock chimneyed above the crown of the tunnel a distance of 15 to 20 ft (4.6 to 6.0 m) and stalled the TBM. The shear zone was approximately 30 ft (9 m) wide and included zones of fast raveling to cohesive running ground.

After stabilization of the ground near Station 21+35, TBM excavation proceeded. TBM penetration rates over the next 2,000 ft (610 m) in massive Pikes Peak Granite proved to be much lower than anticipated. At Station 51+60, a broad stock of Windy Point Granite was encountered in the tunnel. From Station 51+80 to Station 53+95 high water flows were found associated with the stock. The measured flows in this zone within 50 ft (15 m) of the tunnel heading ranged from 261 gpm to 669 gpm (16.5 liters per second to 43.5 liters per second).

Pikes Peak Granite was encountered again at Station 61+30. TBM excavation continued to Station 66+30, where on November 19, 1990, a major shear zone with fast raveling to running ground was encountered. The zone was about 30 ft (9 m) wide. Considerable difficulty was encountered in stabilizing the zone. The TBM excavated through the shear zone near the end of December in 1990.

A second stock of Windy Point Granite was encountered at Station 70+60 extending to approximately Station 86+40. High water inflows were encountered from approximately Station 82+70 to Station 87+70 spanning the transition from the Windy Point Granite Stock to the Pikes Peak Granite. Between January 11, 1991 and April 18, 1991, water inflows within 50 ft (15 m) of the tunnel heading were measured as the tunnel advanced. The flows ranged from 107 gpm to 1,838 gpm (7 liters per second to 122 liters per second). At Station 85+40 a concrete bulkhead was construct to control the water and allow grouting of the water bearing joints ahead of the bulkhead. After the contractor completed some minor grouting, the owner directed the contractor to suspend the grouting and proceed with the TBM excavation.

Major shear zones with considerable soft ground behavior were encountered at Station 101+96 and Station 112+60 in the zone known as the North Field Shear Zone. During the tunnel advance through this area numerous percussion probe holes were drilled ahead of the tunnel heading to evaluate ground conditions.

Tunneling continued through the Pikes Peak Granite reaching the shaft at Station 165+82 in late October of 1991.

The excavation of the Stanley Canyon Tunnel required nearly three years or 36 months, including project shutdowns to stabilize shear zones and handle the water conditions.

GEOTECHNICAL FACTORS IMPACTING TUNNEL EXCAVATION

There were three geotechnical factors that had significant impacts on the Stanley Canyon Tunnel excavation. These include: 1) adverse tunnel ground related to shear zones; 2) adverse ground water inflows; and 3) more difficult boreability properties than anticipated through the competent Pikes Peak Granite, especially between Station 00+84 and Station 125+00.

Shear Zones

Shear zones were anticipated along the tunnel alignment. In fact, the geotechnical investigations targeted shear zones in the first 2,000 ft (610 m) of the tunnel and in the North Field Shear Zone. Based on the investigations, the shear zones were expected to be relatively narrow. Extremely adverse ground behavior was not expected.

In the tunnel, the zones of completely crushed and altered rock were up to 30 ft (9 m) wide. Although ground water inflows from the shear zones were not excessive, high seepage pressures or ground water gradients resulted in the fast raveling to cohesive running ground behavior of the mechanically crushed rock. The project TBM was not designed to efficiently handle the adverse conditions. The open cutters and domed shape of the cutter head allowed failure of the face and crown over and in front of the machine when the broader shear zones were encountered. Stabilizing the zones around the TBM was difficult and time consuming. Hard granite encountered on the other side of several shear zones resulted in gripping problems as the TBM advanced. The shear zones resulted in three major project delays.

Adverse Ground Water Flows

The Windy Point Granite Stock was the source of the adverse ground water flows that were encountered in the Stanley Canyon Tunnel. Flush flows up to approximately 675 gpm (43 liters per second) were encountered upon first entering the eastern limb of the Windy Point Granite Stock, and flush flows up to approximately 1,100 gpm (69 liters per second) were encountered upon leaving the western limb of the stock. The Windy Point Granite Stock basically acts as an aquifer with large volumes of water in storage, especially in the very open jointed and apparently de-stressed rock near the margins of the stock. This was demonstrated by peak tunnel flows of over 4,000 gpm (252 liters per second) with sustained tunnel flows of more than 1,500 gpm (95 liters per second) more than one year after excavation through the stock. In other sections of the tunnel, the ground water inflows were relatively minor, as anticipated based on the pre-construction geotechnical investigations.

The ground water flows significantly impacted the tunnel construction. The peak tunnel flows exceeded the capacity of the contractor's water handling system, washing pipelines, and settling basins down the slope at the tunnel portal. The water handling system had to be redesigned for much higher flows. The large sustained flows slowed every phase of the construction. In preparation for lining the tunnel, an additional pipeline was constructed to convey flows to the portal and the water handling system.

Difficult Boreability of Pikes Peak Granite

As the excavation of the Stanley Canyon Tunnel proceeded from approximately Station 21+35, it was recognized that the excavation rates in the Pikes Peak Granite were much lower than anticipated based on the pre-construction geotechnical investigations. In areas of good tunnel ground, not affected by shear zones, unstable ground conditions, or excessive water inflows, TBM IPR's were between 7 to 8 ft per hour (2.1 to 2.4 m per hour), approximately 60 percent less than the rates anticipated. However, TBM excavation rates in the high strength Windy Point granite proved to be slightly higher than anticipated. Comprehensive, post-excavation geotechnical investigations were performed to thoroughly evaluate the tunnel rock properties, particularly as they relate to boreability.

POST-EXCAVATION GEOTECHNICAL INVESTIGATIONS

Geologic Mapping

The entire tunnel was geologically mapped at a scale of 1 in = 10 ft. Geologic factors documented included rock type, rock classification, joint orientation, joint spacing, weathering, alteration, water condition, shear zones, and other significant geologic features.

Core Sampling

Core sampling was performed in order to obtain samples for laboratory testing. Systematic core sampling was completed on 500-ft (152 m) centers along the tunnel. In addition to the systematic samples, additional core samples were obtained from the rock where the TBM excavation data indicated high propel pressures with low IPR's.

The coring was done using a portable coring machine. The core borings averaged approximately 2 ft (0.6 m) deep into the side walls of the excavated tunnel. The core samples were logged as the drilling progressed and were described for grain size, microfractures (if visible), joints, weathering, and other pertinent rock properties. Core orientation was noted and marked with a chisel. Suitable intact samples were selected for laboratory testing.

Intact Physical and Engineering Property Testing

A total of 105 core samples from the Stanley Canyon Tunnel were tested for intact physical and engineering properties. The testing included 105 density tests, 105 unconfined compressive strength tests, measurement of modulus of elasticity for 75 core samples, and 49 Brazilian tensile strength tests. All the work was performed in accordance with ASTM procedures. The average results of the laboratory testing are presented in Table 1. The average intact properties from the pre-construction investigations are provided for comparison.

Petrographic Analysis

Thin sections were prepared from 25 samples and analyzed petrographically. The petrographic work on the thin sections included mineral point counts to determine mineral percentages, point count scans of mineral grain boundaries, and evaluation of microfracturing. Photomicrographs were prepared for selected samples.

Boreability Testing

Tests were conducted at the Colorado School of Mines Earth Mechanics Institute (EMI) to investigate the boreability properties of the rock. The tests included punch indentation tests, cerchar abrasivity tests, and linear cutting tests. The punch indentation and cerchar abrasivity tests were conducted on 6-in (0.15 m) diameter core samples obtained from the tunnel walls. Large block samples of Pikes Peak Granite and Windy Point Granite were excavated from the tunnel wall by line drilling and blasting for the linear cutting tests.

Table 1. Average Intact Mechanical Rock Properties

	Pre Construction Data				
	Pikes Peak Granite 00+00 to 125+00	Pikes Peak Granite 125+00 to 165+82	Windy Point Granite	Pikes Peak Granite	Windy Point Granite
Unit Weight					
pcf	164.7(65)	163.8(27)	165.3(13)	162.1(29)	162.6(9)
g/cc	2.64	2.63	2.65	2.60	2.61
Unconfined Compressive Strength					
psi	22,055(52)	17,463(15)	25,321(12)	15,365(17)	25,167(6)
MPa	152	120	175	106	174
Youngs Modulus E Tan^{50}					
10^6 psi	9.50(35)	8.40(15)	10.1(11)	6.26(12)	6.00(3)
10^4 MPa	6.55	5.79	6.96	4.31	4.14
Brazilian Tensile Strength					
psi	1,191(26)	1,085(15)	1,575(8)	1,026(7)	1,299(2)
MPa	8.2	7.5	10.9	7.1	9.0

Note: (15) indicates 15 samples tested.

ANALYSIS OF INTACT PHYSICAL AND ENGINEERING PROPERTIES AND PETROGRAPHIC STUDIES

Analysis of the laboratory testing and the petrographic studies of core samples obtained from the Stanley Canyon Tunnel indicate the Pikes Peak Granite has been metamorphosed by intrusion of the Windy Point Intrusive Stock. The rock becomes petrographically different, denser, and stronger the closer the rock in the tunnel is to the Windy Point Intrusion. These differences in the rock properties resulted in a stronger, tougher rock than anticipated.

Figure 2 is a geomechanical profile of the tunnel. Factors shown on this figure include unconfined compressive strength, density, and the nature of crystalline grain boundaries plotted with location along the tunnel profile. The unconfined compressive strength of the Pikes Peak Granite increases as the tunnel approaches the Windy Point Stock and drops off in strength away from the stock toward the end of the tunnel. This relationship is also illustrated on Figure 3, which is a plot of unconfined compressive strength versus distance from the Windy Point Stock. A break in the curve clearly occurs at a distance of 4,000 to 5,000 ft (1,219 to 1,524 m) away from the Windy Point Stock.

The petrographic analysis indicates the mineral grain boundaries of the Pikes Peak Granite become more interlocking as the distance to the Windy Point Stock decreases. This relationship is plotted on Figure 2. The relationship of percent of interlocking grain boundaries versus

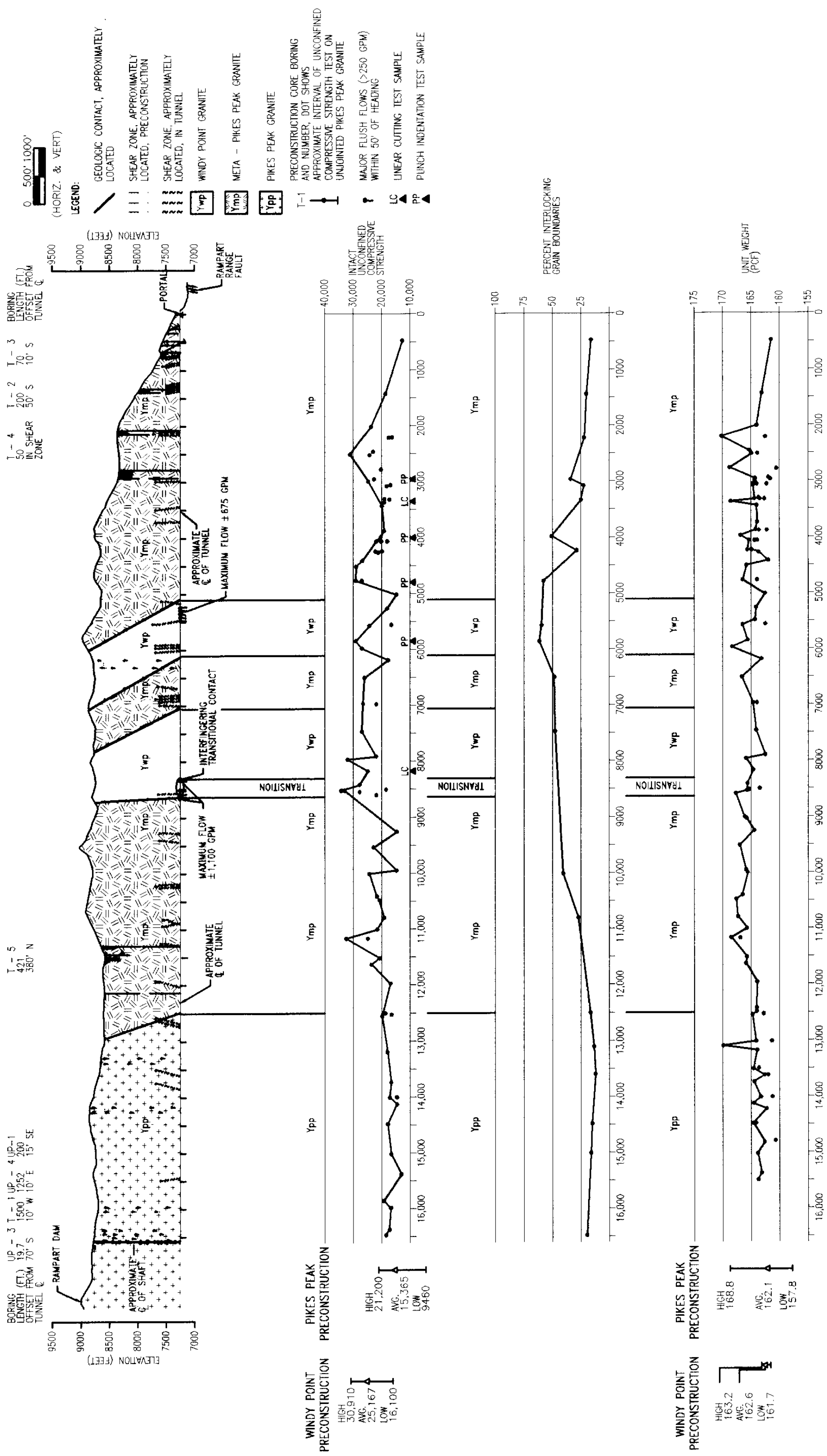

Figure 2. Geomechanical Profile, Stanley Canyon Tunnel

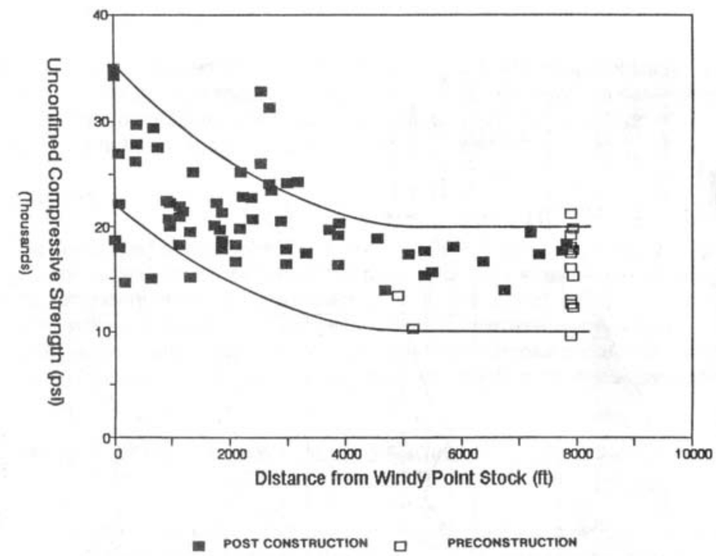

Figure 3. Pikes Peak Intact Compressive Strength versus Distance from the Windy Point Stock.

distance from the Windy Point Stock is illustrated in a different format on Figure 4. A clear break in the curve occurs again at a distance of 4,000 to 5,000 ft (1,219 to 1,524 m) away from the Windy Point Stock.

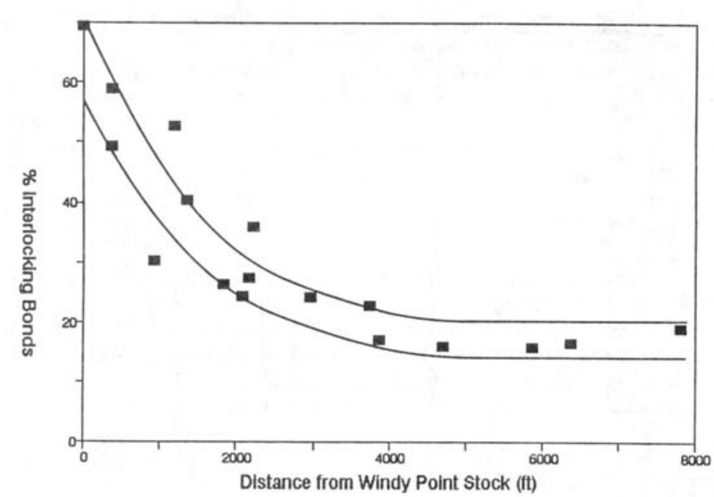

Figure 4. Pikes Peak Percent Interlocking Mineral Bonds versus Distance from the Windy Point Stock.

The contrast in the mineral boundaries in the Pikes Peak Granite from those described in the pre-construction documents to those found during excavation of the tunnel are quite pronounced. Figure 5 is a tracing of the mineral boundaries from a photomicrograph of the Pikes Peak Granite from Bore Hole T-1 drilled at the shaft for the pre-construction geotechnical investigations.

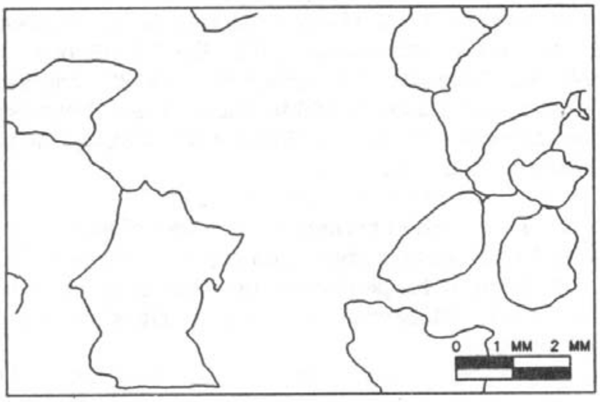

Figure 5. Tracing of Photomicrograph Mineral Boundaries, Pikes Peak Granite Pre-Construction Investigations T-1 at 972.5'.

Note that the rock is very coarse-grained and the grain boundaries are predominantly smooth, linear to cuspate. Figure 6 is a tracing of the mineral boundaries from a photomicrograph of Meta-Pikes Peak Granite from Station 100+06. The photomicrograph shows a less coarse-grained granite with well developed, sutured, interlocking grain boundaries, especially between the quartz, the feldspar, and the biotite mineral grains.

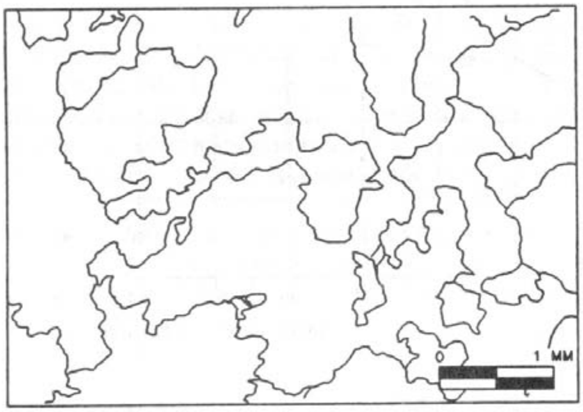

Figure 6. Tracing of Photomicrograph Mineral Boundaries, Pikes Peak Granite Tunnel Station 100+06.

The strength difference of the rock is also quite dramatic. The average unconfined compressive strength measured for the Pikes Peak Granite for the pre-construction geotechnical investigations was approximately 15,365 psi (105 MPa), ranging from 6,120 psi (42 MPa) to 21,200 psi (146 MPa). The average unconfined compressive strength for the Mete-Pikes Peak Granite from Station 00+00 to Station 125+00 sampled for the post-excavation geotechnical investigations was 22,055 psi (152 MPa), ranging from 13,875 psi (96 MPa) to 34,692 psi (241 MPa). Figure 7 illustrates the relationship between the percent of interlocking grain boundaries in the granite samples to sample unconfined compressive strength. The plot shows a strong correlation.

Statistically, this plot has a coefficient of correlation "R" of 0.89. The metamorphism of the Pikes Peak Granite resulted from stress and thermal effects caused by the Windy Point intrusion.

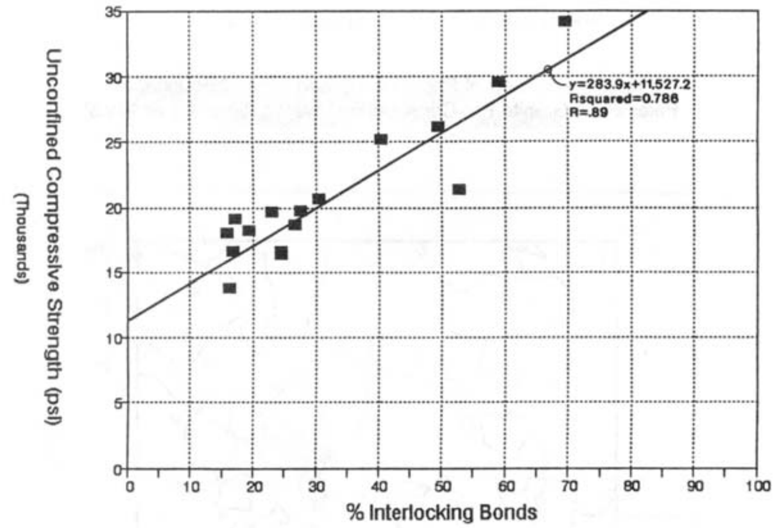

Figure 7. Pikes Peak Percent Interlocking Mineral Bonds versus Intact Unconfined Compressive Strength

The knitted rock fabric also results in a denser rock mass. The rock becomes measurably denser approaching the Windy Point Stock. The relationship of density or unit weight versus distance from the Windy Point Stock is shown on Figure 2.

Beyond approximately Station 125+00 the Pikes Peak Granite showed little metamorphic effects from the intrusion of the Windy Point Stock. The mineral boundaries were smooth to cuspate, the rock density decreased measurably, and the unconfined compressive strength averaged 17,483 psi (120 MPa), ranging from 13,875 psi (98 MPa) to 2,024 psi (139 MPa).

The contact metamorphism, which resulted in increased strength of the metamorphosed zone of the Pikes Peak Granite, was not detected in the pre-construction geotechnical investigations. The pre-construction borings were located mainly to investigate the geotechnical conditions of the rock at the shaft, at the portal, and at shear zones. The shaft lies outside the zone of metamorphism caused by the Windy Point Granite intrusion, and the portal is near the margin of the metamorphism caused by the Windy Point Granite intrusion. Most of the intact specimens tested for the pre-construction investigations came from borings drilled at the shaft.

BOREABILITY INVESTIGATIONS

Boreability testing conducted during construction included linear cutting tests of large block samples of the metamorphosed Pikes Peak Granite and the Windy Point Granite, punch indentation tests of 6-in (0.15 m) diameter cores of Meta-Pikes Peak Granite and the Windy Point Granite, and cerchar abrasivity index (CAI) tests of the granites to estimate cutter wear.

Linear Cutting Tests

The linear cutting machine (LCM) is a laboratory test machine designed to provide data to evaluate the boreability of rock and to develop TBM performance estimates. The LCM has been used extensively over the last two decades to predict field boreability. It is capable of closely simulating field cutting conditions by allowing the use of full-scale cutter bits. The machine can generate the entire range of field cutter loads and penetrations while allowing the testing and evaluation of different cutter spacings, cutter traverse velocities, and other operational parameters related to TBM boring performances. As a result of these capabilities, LCM testing is designed to eliminate any potential problems related to scaling effects between the laboratory and field bit performance. The accuracy of the results produced from the LCM tests has been confirmed through comparison with field TBM performance date collected from numerous tunnels bored in different rock formations and ground conditions. Figure 8 is a schematic of the LCM.

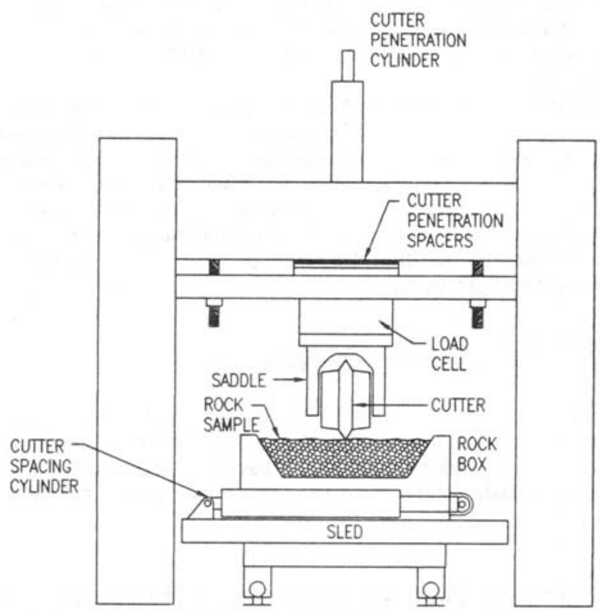

Figure 8. The Linear Cutting Machine (LCM).

A series of tests were conducted on the large blocks of Meta-Pikes Peak Granite and Windy Point Granite. The results of the linear cutting tests were used to prepare machine performance estimates for the Stanley Canyon Tunnel. Analysis of the data indicated the following:

1. The machine performance estimates based on the results of linear cutting tests were found to agree very closely with the actual machine performance at the tunnel locations where the test samples were obtained.

2. For the Windy Point Granite, the specific energy of cutting reached its minimum at a penetration of about 0.10 in (0.254 cm). In Pikes Peak Granite, the required depth of penetration for the specific energy to attain its minimum value was around 0.20 in (0.508

cm). This indicates that the Pikes Peak Granite required much deeper cutter penetrations before efficient chip formation could commence. The optimum spacing to penetration (s/p) ratio for the Pikes Peak Granite was much lower than the Windy Point Granite. The optimum s/p ratio for Pikes Peak Granite was found to fall within the range typical of rocks showing little brittleness in response to mechanical excavation.

3. The cuttings generated from LCM tests in Pikes Peak Granite included a higher percentage of fines than the Windy Point Granite. This resulted in a smaller size distribution for the Pikes Peak cuttings. The occurrence of more fines was also confirmed by visual observations of the rock surface during testing. An extensive amount of crushed material was seen to form in the cutter path.

4. The evaluation of all test data and observations leads to the conclusion that the Pikes Peak Granite lacks the level of brittleness required for efficient rock failure, particularly at low cutter penetrations. The rock appears to absorb a significant amount of cutting energy prior to chip formation. Because of these characteristics, it was found to cut harder than its compressive strength would indicate. Hence, TBM performance predictions developed based on rock strength data alone were found to overestimate the actual machine performance by about 25 to 30 percent.

Punch Indentation Tests

In addition to linear cutting tests, a series of punch indentation tests were also performed on rock samples retrieved from the Stanley Canyon Tunnel. The indentor used was a cut-off segment of a cutter ring used on the Stanley Canyon TBM. The indentations were spaced 2.75 in (0.07 m) apart to duplicate field cutter spacing. A million-pound capacity stiff testing machine was used to carry out the indentation tests. For each indentation, the force versus penetration was measured and used to calculate the penetration index. This index was then used to determine the TBM penetration rate at a given cutter load. The boreability estimates based on the indentation tests are summarized in Tables 2 and 3.

Table 2. Actual, Predicted, and Calculated TBM Performance in the Meta-Pikes Peak Granite.

Rock Type	Tunnel Station	Type of Measurement	IPR (ft/hour)	IPR (m/hour)
Pikes Peak	39+95, 47+90	Punch Indentation	7.43	2.26
Pikes Peak	33+40	Linear Cutting	7.40	2.25
Pikes Peak	00+00, 125+00	Actual Field	7.42	2.26
Pikes Peak	Whole Tunnel	Predicted*	13.30	4.05

* Based on pre-construction geotechnical investigations.

Table 3. Actual, Predicted, and Calculated TBM Performance In the Windy Point Granite.

Rock Type	Tunnel Station	Type of Measurement	IPR (ft/hour)	IPR (m/hour)
Windy Point	58+60	Punch Indentation	7.43	2.26
Windy Point	81+85	Linear Cutting	7.70	2.34
Windy Point	Whole Tunnel	Actual Field	9.71	2.96
Windy Point	Whole tunnel	Predicted*	8.30	2.53

* Based on pre-construction geotechnical investigations.

Cercher Abrasivity Index Tests

A series of CAI tests were performed to develop data for estimating cutter costs. Based on the results of CAI measurements, the cutter cost estimates for the Pikes Peak Granite and the Windy Point Granite were \$5.50 and \$7.30 per yd^3 (\$7.15 and \$9.99 per m^3) of material excavated, respectively. For the Pikes Peak Granite, the cutter costs derived from laboratory abrasivity determinations were higher than those predicted based on the pre-construction geotechnical investigations. The higher than predicted cutter wear is attributed to extra rock crushing occurring in the cutter path due to spongy behavior of Pikes Peak Granite. However, cutter costs for the Windy Point Granite based on the pre-construction geotechnical data were found to be in close agreement with the CAI laboratory estimates.

Boreability Analysis

The linear cutting tests and punch indentation tests accurately reproduced the actual TBM performance achieved in the Stanley Canyon Tunnel. Table 2 summarizes the actual TBM performance, calculated performance based on boreability testing, and predicted performance using the EMI model based on pre-construction geotechnical data for the Pikes Peak Granite. Table 3 provides the same summary for the Windy Point Granite.

A summary of the field performance of the Stanley Canyon TBM for the various rock types encountered is presented in Table 4. The table shows IPR's attained by the TBM normalized from machine propel pressures of between 4,000 and 4,500 psi (28 MPa and 31 MPa). The best penetration rates were attained in the Windy Point Granite, the rock type with the highest unconfined compressive strength. The Windy Point Granite exhibited brittle behavior, which resulted in efficient chipping of the rock and the rock was generally more closely jointed which also improved boreability.

The metamorphosed Pikes Peak Granite between Station 00+00 and Station 125+00 was the most difficult to bore with average penetration rates for the entire tunnel in massive granite of 7.4 ft (2.25 m) per hour. This is the same rate predicted by the linear cutting tests of the metamorphosed Pikes Peak Granite.

Table 4. Summary of TBM Penetration Rates versus Rock Type Propel Pressure of 4,000 to 4,500 psi (28 MPa to 31 MPa).

Rock Type	IPR (ft/hour)	IPR (m/hour)
Pikes Peak 00+00 to 125+00	7.42	2.26
Pikes Peak 125+00 to Shaft	7.92	2.35
Windy Point	9.71	2.96

The unmetamorphosed Pikes Peak Granite between Station 125+00 and Station 165+82 was also difficult to bore with an average IPR of 7.72 ft (2.35 m) per hour. This appears to be due in a large part to the coarse-grained texture of the Pikes Peak Granite and lack of brittleness. Another important factor was that at this stage in the project, the TBM was showing the effects of the difficult boring conditions. Machine wear may have contributed to the low advance rates.

SUMMARY AND CONCLUSIONS

The project provides an interesting case history of geotechnical problems that can occur during the construction of a hard rock tunnel having relatively simple geologic conditions. The comprehensive geotechnical investigations conducted during construction allowed in depth study and analysis of the conditions. The geotechnical problems that were encountered, however, can be explained by understanding of the project geologic setting.

Shear Zones

The geologic setting was near a major mountain frontal fault. Numerous shear zones were expected. Shear zones often behave simultaneously as ground water drains and ground water barriers. Permeability parallel to the shear zones is usually much higher than permeability perpendicular to the shear zones. This is because seams of crushed rock within the shear zones often have been altered to clay. As the TBM approached the shear zones, they did not immediately drain to the tunnel face, but maintained a higher ground water head than the unsheared rock. When the TBM penetrated the shear zones, a relatively high ground water gradient was encountered between the sheared rock and the tunnel face. The gradient exceeded the critical gradient, which resulted in raveling and running of the sheared rock materials which stalled and seized the TBM.

High Ground Water Flows at the Margins of the Windy Point Stock

Cooling at the margins of intrusive igneous bodies like the Windy Point Stock often results in a de-stressed, open jointed zone caused by thermal contraction. This zone in the Stanley Canyon geologic setting behaved as an aquifer with high joint permeability and high sustained ground water flows.

Difficult Boreability

Coarse-grained granites like the Pikes Peak Granite are difficult to bore because they tend to be tough, and absorb energy, causing crushing of minerals and generation of excessive abrasive fines, as opposed to brittle fracture and efficient chipping of the rock. The boreability is much more difficult than the compressive strength would indicate.

Metamorphism of the Pikes Peak Granite by Intrusion of the Windy Point Stock

Metamorphism of the Pikes Peak Granite compounded the difficult boreability because of the interlocking of the mineral grains and increased rock strength. The temperature and pressure effects associated with igneous intrusions similar to the Windy Point Stock often result in significant changes to the host rock. The changes have important impacts on the host rock engineering properties.

In hindsight, there is little doubt that a more modem, versatile TBM, designed to handle a wide range of rock conditions could have more efficiently excavated the Stanley Canyon Tunnel. A machine with higher thrust and torque would have had higher advance rates in the Pikes Peak Granite. A fully shielded machine with a flat closed cutter head and recessed cutters, and versatile gripping capabilities would have had an easier time excavating through the shear zones.

Lessons Learned

What information, in addition to the pre-construction geotechnical investigations, would be required for an owner to specify, or a contractor to structure his bid around a more powerful and versatile TBM on a project similar to Stanley Canyon?

1. Boreability Testing: Boreability testing, in particular the punch indentation tests and linear cutting tests, accurately reproduced the actual TBM excavation rates achieved in the Stanley Canyon Tunnel. Boreability testing should be considered for pre-construction geotechnical investigations for hard rock tunnels to provide direct information on rock excavation properties.

2. Engineering Geologic Insight Based on Experience: Engineering geologic insight requires understanding the geologic setting and interpreting the potential impacts the geologic conditions may have on the project. The pre-construction geotechnical investigations for the Stanley Canyon Tunnel reasonably documented the geologic setting. The potential impacts of the geologic conditions on the excavation of the tunnel were not recognized.

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NOTE: This paper was originally presented at the Rapid Excavation and Tunneling Conference, San Francisco, California, June 18-21, 1995. We are grateful to SME (www.smenet.org) for allowing us to republish this paper. Boreability testing for hard rock TBM projects has now become common as part of the pre-bid geotechnical investigations.

TUNNEL TRILOGY -
AN ILLUSTRATION OF CONSTRUCTION, INSPECTION AND MAINTENANCE PROJECTS UTILIZING TUNNELS

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Key Terms: tunnel, tunneling, construction, inspection, maintenance, Denver Water, Foothills, Roberts, Moffat

ABSTRACT

Water, or the lack of it, has been a major concern of the Denver area since its humble beginnings as a collection of cabins and teepees along the banks of the South Platte River and Cherry Creek. The Denver Water Department's vast system of dams, tunnels, treatment plants, conduits and pump stations was developed to serve the need for water supply to the semi-arid Denver environment. Critical components of the water distribution system are the tunnels. Construction, inspection and maintenance projects are briefly summarized for these critical components and are illustrated utilizing three tunnels in the Denver Water system: 1) Foothills Tunnel, 2) Roberts Tunnel and 3) Moffat Tunnel. Tunnel locations are shown on Figure 1.

FOOTHILLS TUNNEL

Construction is the process of building a new facility, i.e. tunnel, water treatment plant, dam or building etc. or adding parts to existing facilities through upgrading and remodeling. This process includes planning, feasibility studies, exploratory drilling, geologic mapping, and engineering and documentation of the actual construction.

GEOLOGY OF THE FOOTHILLS TUNNEL

Introduction. The Foothills tunnel was built to divert water from Strontia Springs dam and reservoir to the Foothills Treatment Plant, a water treatment facility. The plant and the east portal of the tunnel lie within the tilted sedimentary rocks forming valleys and hogbacks along the east side of the Front Range. The tunnel penetrates approximately 4,600 feet of sedimentary rocks and 13,300 feet of crystalline metamorphic and igneous rock for a total length of 17,935 feet. The sedimentary section of the tunnel was mined with a tunnel boring machine (TBM); the igneous and metasedimentary sections were mined using conventional drill and blast methods.

Geologic Setting. The Strontia Springs area and adjacent areas are located on the eastern slope of the Colorado Front Range. The core of the range is a complex of crystalline Precambrian igneous and metamorphic rock locally intruded by dikes and stocks of Tertiary igneous rocks. The core is flanked on the east and west sides by sedimentary rocks that generally dip away from

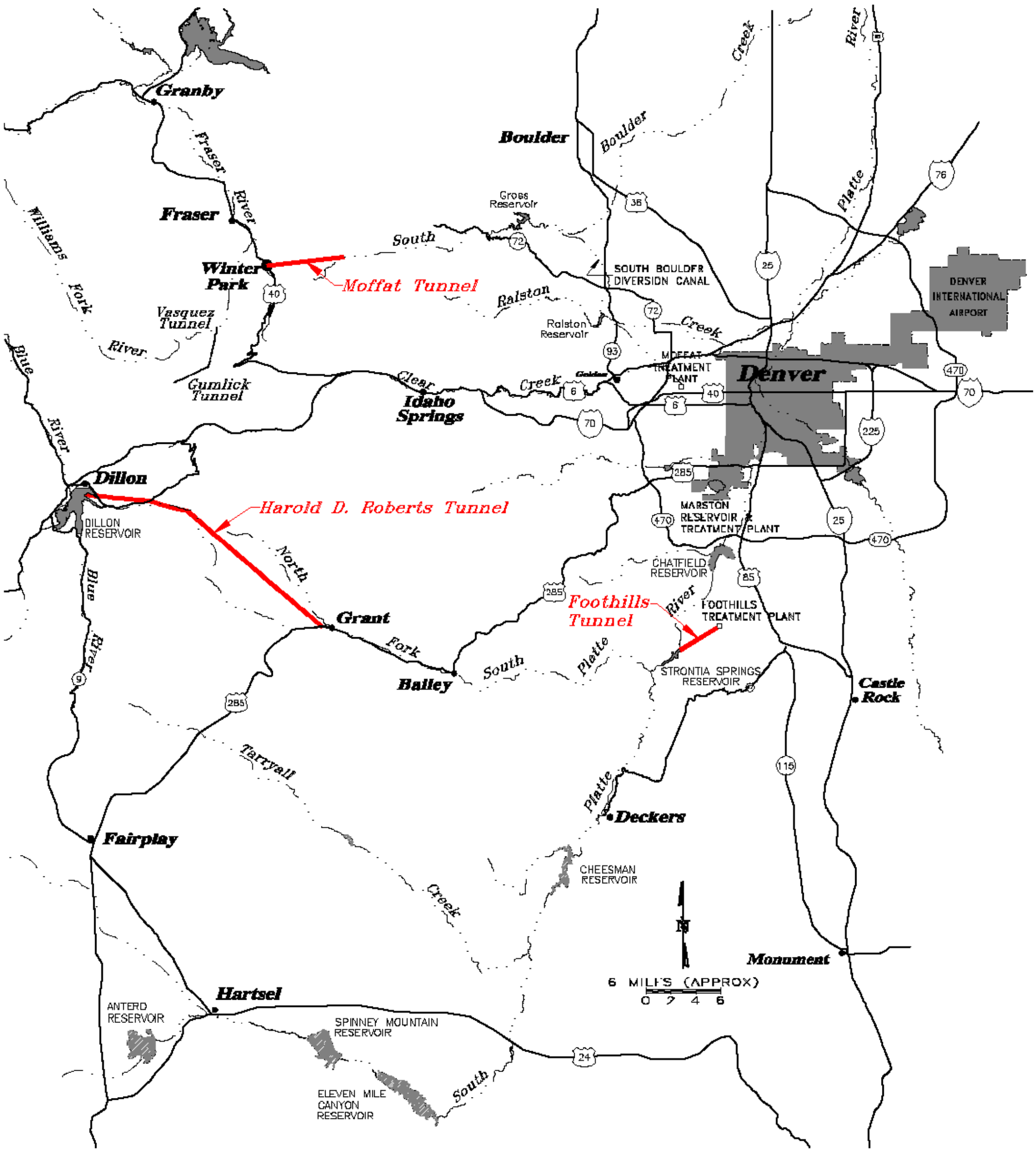

Figure 1. Location Map Showing the Foothills, Roberts and Moffat Tunnels of the Denver Water Supply System.

the core. Numerous faults that range in age from Precambrian to Recent intersect and displace bedrock.

The Strontia Springs dam and reservoir are entirely in crystalline rock. The Foothills Tunnel intersects both crystalline and sedimentary rocks and passes through the contact between layered Paleozoic sedimentary rocks resting unconformably on ancient crystalline rocks, that consist mostly of layered metamorphic schists and gneisses.

Sedimentary Rocks. The sedimentary portion of the Foothills tunnel was portalled in the Greenhorn Limestone and penetrated down the stratigraphic section through the Graneros Shale, Dakota Group, Morrison Formation, Ralston Creek Formation, Lykins Formation, Lyons Sandstone and Fountain Formation. These rocks range from Cretaceous to Pennsylvanian in age.

The sedimentary rocks penetrated by the tunnel include coarse-grained arkosic sandstone with some thick beds of siltstone and shale, crossbedded quartzose sandstone, interbedded siltstones and mudstones with minor thin beds of sandstone and limestone, massive quartzose sandstone containing thick shale units, limely shales and limestone. The sedimentary rocks strike approximately N.30°W. and dip on the average 60° to the northeast. The contact between the sedimentary rocks and the crystalline rocks is an erosional uncomformity. In the tunnel, the contact dips steeply (85°-90°) and is tight and dry, with no evidence of faulting along it.

Crystalline Rocks. The crystalline rocks in the Foothills Tunnel consist of metasedimentary gneisses and schists, and amphibolite. These rocks were invaded by granitic solutions that produced complexly folded masses of migmatite, granitic gneiss, aplite, and large bodies of pegmatite.

The crystalline rocks of the tunnel can be grouped into three dominant types: (1) migmatite; (2) biotite gneiss; and (3) granitic gneiss. However, a variety of other rock types were observed during mapping that include: granite, aplite, pegmatite, granitic gneiss, gneissic granite, biotite gneiss, garnet-biotite gneiss, sillimanite-biotite gneiss, gneissic marble, quartzite, diorite, and amphibolite. Contacts between rock types are generally gradational. Amphibolite occurs in scattered pods of various sizes; pegmatite occurs in dikes and pods predominantly conformable with the foliation.

Three mafic andesite and/or basalt dikes of Cretaceous or Tertiary age cut the metamorphic rocks of the tunnel. One of the dikes, located between tunnel stations 109+00 and 108+00, intersects metamorphic rocks and trends northeast. This dike shows considerable propylitic alteration, but no metamorphism. The fracture surfaces in the dike, for the most part, are slickensided. The other dikes, classified as basalts, are located approximately 1,000 feet west of the crystalline/sedimentary contact and trend northwest. They exhibit deuteric alteration, but no effects of metamorphism.

Sandstone Dikes. Several thin sandstone dikes are primarily associated with the Stevens Gulch and Mill Gulch shear zones. The fabric and mineralogy of these dikes are similar to that of other numerous sandstone dikes intersecting the crystalline rocks of the Front Range. The dikes are brecciated in part along their perimeters. It is supposed that the dikes were derived from basal

Paleozoic quartz sandstones (probably the Cambrian Sawatch sandstone) and were deposited in cracks in a passive, non-tectonic manner.

Joints. The crystalline rocks of the Foothills Tunnel exhibit two well-developed joint systems. One set strikes N.40°-75°W. and dips 55°-75°S.W.; the other set strikes N.20°W. to N.10°E., mostly dipping steeply to the west (about 75°), but also rolling so as to dip steeply to the east. Several minor joint sets as well as random joints were also recorded during tunnel mapping. Few joints planes are visible in some of the intensely sheared sections of the tunnel, particularly in the Steven's Gulch and Mill Gulch shear zones. Generally, the joints are tight with major calcite crusts in fresh rock, but exhibit varying amounts of iron-stained clay coatings near and in sheared zones.

Foliation. The foliation of the igneous and metasedimentary rock predominantly trends N.75°W. and dips 60°-80°N.E.; however abrupt changes in strike and dip occur locally. The foliation of the rock in the tunnel is well-developed and conforms to the geologic mapping of foliation of the Strontia Springs dam and reservoir site as well as the regional structural pattern defined by previous work. Most of the shearing mapped in the tunnel is foliation controlled, including the two major shear zones--the Steven's Gulch and the Mill Gulch zones.

Shear Zones and Faults. Two major shear zones intersect the Foothills Tunnel. The Steven's Gulch shear zone is located at tunnel stations 150+85 to 155+50; the Mill Gulch shear zone is located at tunnel stations 96+50 to 91+50. Both are foliation controlled shear zones and faults that are thought to be of Precambrian age and reactivated during the Laramide orogeny of the Late Cretaceous time. They trend northwest, correlating with the regional trend of faults mapped at the surface. A third significant shear zone is located from tunnel station 176+50 to the west end of the tunnel at the intake tower. All three zones contain clay gouge along numerous shears and zones of highly crushed and altered, iron-stained, friable, gougy rock. Although steel supports and lagging were used throughout these sheared intervals, they did not present significant problems during excavation.

Several foliation-controlled shear zones of varying width occur throughout the tunnel. The faults and shears are related to the overall competency of the rock. There are numerous small shears about two inches wide occurring locally along biotite-rich bands in otherwise fairly fresh, competent rock. These shears had no significant effect on the stability of the tunnel during or after excavation.

Offset along faults in the tunnel was difficult to determine due to the complex and varied fabric of the metamorphic rocks as well as the exposures of movement planes being limited to the tunnel diameter (14.5 feet). However, careful examination of fault planes show slickensides that indicate strike-slip, dip-slip, normal, and reverse movement. Occasionally, the slickensides indicated more than one time or direction of movement. The predominant direction of movement along faults and slip planes in the Foothills tunnel is strike-slip.

Summary. Geotechnical investigations in the Foothills and Aurora-Ramparts water diversion tunnels, at the Strontia Springs dam and reservoir site, and at the proposed Turkshead and Two Forks dam sites in South Platte Canyon southwest of Denver, Colorado, have yielded a very large volume of detailed geologic data in an area that is typical of general geologic conditions in many portions of the eastern slope and foothills of the Colorado Front Range. Data obtained from these studies not only have scientific value in the interpretation of the results of regional geologic and geophysical investigations, but also will be useful in planning and completion of geotechnical studies elsewhere in the Front Range.

Correlation of fault zone and shear zone attitudes with foliation in the metamorphic rocks indicates that for many of the large and for many of the minor faults and shear zones, the dislocations occurred along multitudes of small slip surfaces parallel to the foliation and not along single, sharply defined surfaces. Moreover, in larger faults cutting across bedrock structures, movement has taken place along a host of small, parallel to subparallel shear surfaces in zones up to several hundred feet wide. In general, mass movements during regional dislocations of bedrock appear to be more a consequence of slight offsets along a myriad of discontinuous fractures than a result of larger offsets along single, laterally and vertically continuous fractures. Most of the faults, then, are accurately described as "shear zones."

The primary permeability of the crystalline rocks of the Front Range is essentially zero. Movement of groundwater through these rocks is confined to interconnecting fractures which provide secondary permeability. Most of the fractures in bedrock are tight and, in many places, filled with clayey gouge. Tests at the Strontia Springs dam site, located near the Foothills Tunnel, indicate that the foundation rock, including the joints, faults, and shear zones, in general has a low permeability near the surface. Permeability of the rock mass is exceptionally low at depths of 100 to 300 feet and probably is non-existent at depths greater than 300 to 400 feet.

Observations in the Foothills Tunnel shows that penetration of groundwater below the earth's surfaces is localized by infrequent cracks associated with in or adjacent to fault and shear zones and that penetration of groundwater along the fault and shear zones, especially where gouge is present, is negligible.

CONSTRUCTION OF THE FOOTHILLS TUNNEL

Construction of Foothills Tunnel, and the associated Conduit No. 26, was carried out by Contractor Shank-Artukovich. The consulting engineer for the tunnel was DMJM/Phillips, Resiter, Haley. Work started on the project in June, 1979, and on the tunnel excavation in September, 1979. Work in the tunnel, including the concrete lining, was completed in June, 1981, and all work on the project was completed in September, 1982. The tunnel was filled with water in December, 1982.

The tunnel was driven from two portal locations using two different tunneling methods, a Calweld Tunnel Boring Machine (TBM) was used to drive 4,600 ft. of tunnel from the East, or downstream, portal, through sedimentary rock. The remaining 13,200 ft., through Precambrian rock, was driven from the Stevens Gulch portal using conventional drilling and blasting.

The fully shielded Calweld TBM excavated a 13.33 ft. diameter tunnel and was equipped to push off the ring beam and lagging support system using 10 ft. - 6 inches diameter x 60 inch stroke hydraulic cylinders operating at pressures up to 8,000 psi (Figure 2).

Sedimentary rocks in the project included sandstones having compressive strengths up to 12,000 psi, and a variety of shales and siltstones having much lower strength. TBM penetration rates varied from 1 inch to 6 inches per minute, with the most common rates being 3-4 inches per minute. Average production was approximately 80 ft. per day, fully supported with ring beams and lagging, working 2 x 8 hour shifts. The graveyard shift was used to maintain the TBM and to advance utilities (Figure 3).

Figure 2. TBM and Trailing Gear at Portal.

Figure 3. TBM Operator.

The remaining 13,200 ft. of tunnel was driven both ways from a canyon, Stevens Gulch, which intersected the tunnel line, approximately 1,700 ft. downstream from the intake in Strontia Springs Reservoir. The 1,700 ft. west leg was driven first, then the 11,500 ft. east leg was driven to hole through with the already completed TBM section. The drilled and blasted tunnel was excavated as a 14 x 14 ft. horseshoe (Figure 4).

Figure 4. Placement of Steel Sets and Wood Lagging.

The Precambrian rocks excavated in the drilled and blasted tunnel included, primarily, migmatite, granitic gneiss, and biotite gneiss. Compressive strengths of these rocks ranged up to 30,000 psi, with an average of approximately 18,000 psi. Two major fault zones, having a total length of approximately 1,000 ft., were encountered. These zones were relatively dry and caused few problems in the tunnel excavation. Mucking out was by a Conway 75 electric mucker loading into 5 cubic yard muck cars.

The final lining of the tunnel included 3,700 ft. of steel lining, concrete backfilled, and 14,100 ft. of concrete lining, both having a diameter of 10.67 ft (Figure 5 and 6).

A 15 ft. diameter x 300 ft. deep surge shaft was constructed approximately 3,000 ft. from the East Portal (Figure 7).

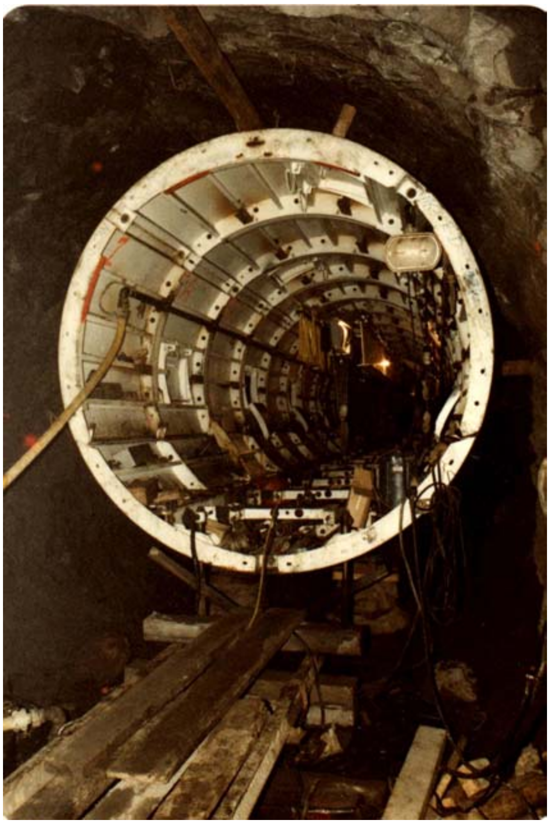

Figure 5. Telescoping Concrete Tunnel Form.

Figure 6. Steel Tunnel Liner.


Figure 7. Surge Chamber.

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ROBERTS TUNNEL

Inspections of an existing tunnel are completed regularly to compile a history of the behavior of the tunnel lining related to detailed construction and geologic information in order to maintain the operational integrity of the tunnel as an important link in the Denver Water system. Inspections provide guidance for tunnel maintenance.

PROJECT DEFINITION AND PURPOSE

Roberts Tunnel is one of the most vital links in the Denver Metro area water supply system; maintaining the tunnel in optimum operating condition is critical for the constant supply of water to Denver. Performance of regular inspections of the Roberts Tunnel compiles a history of the behavior of the tunnel lining related to a detailed review of construction and geologic information. The condition and operational integrity of the tunnel can only be ascertained from the condition of the concrete lining. Inspections provide guidelines for tunnel maintenance. This report provides information of the inspection process of the Harold D. Roberts Tunnel and documents the observations and recommendations of the three tunnel inspections conducted in 1992, 1996, and 1999.

PROJECT BACKGROUND INFORMATION

Post construction inspections of the entire Roberts Tunnel have been completed in 1966, 1992, 1996 and 1999. An inspection of the west portion of the tunnel from the west portal (emergency access shaft in Dillon Reservoir) to the Montezuma Shaft was completed in 1976. Numerous inspections have been made by Denver Water Source of Supply personnel in the areas located from the east portal to the surge chamber and in the vicinity of the bottom of the emergency gate shaft at the west portal during the years the tunnel has been in operation.

DESCRIPTION OF FACILITIES

The Harold D. Roberts Tunnel was constructed to divert water from the Blue River (collected in Dillon Reservoir) on the western slope of the Continental Divide to the North Fork of the South Platte River near Grant, Colorado on the eastern slope of the Continental Divide. The Roberts Tunnel is a pressure tunnel that is lined with concrete and finished to a diameter of 10.25 feet. It is the world's longest major underground water tunnel at 23.3 miles, with a capacity of 1,020 cubic feet per second. At capacity, the tunnel can transport 2,023 acre feet (680 million gallons) of water per day.

Technically, construction on the tunnel lasted for 16 years; however, the contract for the majority of the construction was awarded in 1956, the tunnel was holed through in February 1960, the concrete lining was completed in 1961, and the tunnel began to transport water in 1964. The tunnel was constructed from four headings – one from each portal and two from the access shaft (Montezuma Shaft). The tunnel was mined by conventional methods (drill and blast), supported in fractured and fault zones by steel support ribs and wood lagging, and fully lined with concrete. The concrete lining throughout the tunnel, with few exceptions, is thicker than the design designated. Backfill grouting behind the concrete lining was accomplished throughout the tunnel to ensure complete and uniform transfer of rock load to the lining.

Major features of the tunnel include an intake channel and an intake structure at the West Portal, near Dillon, a 200-foot emergency gate shaft and appurtenant control structures at station 14+40, a 910-foot access shaft at station 459+43 (Montezuma Shaft), a 430-foot surge chamber shaft near the East Portal at station 1231+34, and an outlet works at the East Portal with a hydropower plant 5.5 megawatt capacity generation.

DESCRIPTION OF GEOLOGY

Bedrock Geology. The Roberts Tunnel transects a geologically complex area in the Front Range of the Colorado Rocky Mountain that is made up of Precambrian igneous and metasedimentary rocks, Paleozoic and Mesozoic sedimentary rocks, and Tertiary igneous rocks. Generally, the Precambrian rocks, which constitute the eastern two thirds of the area penetrated by the tunnel, consist of a sequence of metasedimentary rocks that were invaded by igneous rocks. These rocks are predominantly migmatite, microcline gneiss, amphibolite, sillimanite and biotite gneiss, schist and hornblende gneiss intruded by granite. Mesozoic and Paleozoic-age sedimentary rocks are exposed in the western quarter of the tunnel and they include predominantly sandstone, shale, siltstone, limestone, and mudstone. The Tertiary-age intrusive Montezuma quartz monzonite stock constitutes the rock of the central quarter portion of the tunnel. The predominant structural feature of the tunnel is the Williams Range thrust fault. The tunnel also penetrates the Front Range mineral belt, a northeast-trending zone of faults, veins (with associated alteration) and igneous intrusions that extend diagonally across the range. The rock intersected by the tunnel is jointed, folded and contain numerous faults of varied displacements.

Alluvial and Glacial Deposits at Dillon Reservoir. Quaternary-age glacial stream deposits consisting of gravel, sand, and cobbles, that range in thickness from 25 to 40 feet, exist in the Snake, Ten Mile, and Blue River drainages that merge in the Dillon Reservoir area at the west portal of the Roberts Tunnel.

Geology and Tunnel Inspection. The original geologic mapping and knowledge of the conditions in the tunnel during construction are valuable tools for evaluation of conditions in the tunnel lining during inspection and potential maintenance procedures.

Knowledge of the geology including rock type and strength, fracture systems, locations of faults and groundwater inflow zones offer valuable information to predict problem areas as well as aid in solutions for maintenance procedures if necessary.

The possibility that fault movement could damage the tunnel always exists since numerous faults and shear zones occur along the tunnel route. However, the probability that significant damage would occur is very low, and the probability that movement to the extent that it would cut the tunnel completely is extremely low. The geologic record of the Front Range of Colorado shows that fault movement has occurred within the past 20 million years, but recent studies by Denver Water and others for the proposed Two Forks project show that no fault movement has occurred within the past 125,000 years and probably not within the past 500,000 years or longer. The tectonic regime prevailing at the present time in the Front Range is such that it is not likely to produce fault movement that would damage the tunnel within the near future – probably much longer than the expected operational life of the tunnel.

The most fractured section of rock occurs in the western 6.5 miles of the tunnel; this section is predominantly shales, sandstones and siltstones and is also characterized by the Williams Range Thrust Fault, the most dominant structural feature cut by the tunnel. Also, methane gas was detected during mining through the shale zones of the western part of the tunnel. Gas monitoring during the 1992, 1996 and 1999 inspections have not detected methane in the tunnel.

Zones in the tunnel that are potentially susceptible to problems are the fractures, faulting, and alteration associated with the Williams Range Thrust Fault, and the western 6.5 miles of the tunnel. This zone of sedimentary rocks is the most fractured and supported section of the tunnel. Methane gas was reported seeping in the shale during construction.

SCOPE OF WORK

Performance of regular inspections of the Harold D. Roberts Tunnel compiles a history of the behavior of the tunnel lining related to a detailed review of construction and geologic information in order to maintain the operational integrity of the tunnel as an important link in the Denver Water supply system. During the inspection, the inspection team visually determines the condition of the tunnel concrete lining and identifies deficiencies, which may need to be addressed (Figure 8). The inspection team rides the JCI tunnel vehicle (Henry J) through the 23.3 mile tunnel in approximately five (5) hours time (Figure 9). Observations are recorded on a small hand held tape recorder; documentation of tunnel conditions is by representative photographs and video segments, water samples are gathered for water quality analysis and temperature comparisons. Distance traveled in the tunnel and tunnel location are coordinated with an odometer mounted on the Henry J with the brass plate ¼ mile markers located at springline of the tunnel. Prior to the inspection, the geologist reviews the geology and construction conditions in order to refresh knowledge of tunnel areas that are most probably susceptible to problems. The actual physical tunnel inspection is the final product of many months of diligent planning, equipment preparation and tunnel preparation by a large team of talented individuals that form the backbone of the project success. Information that contributes to the planning, equipment operation, safety and success of the Roberts Tunnel Inspection is presented in a separate report.

INSPECTION REPORTS

Pertinent observations are documented in detailed inspection reports. Appendix I includes a summary of three selected Roberts Tunnel inspections. These summaries include the purpose of the inspection, observations, and recommendations for future work.

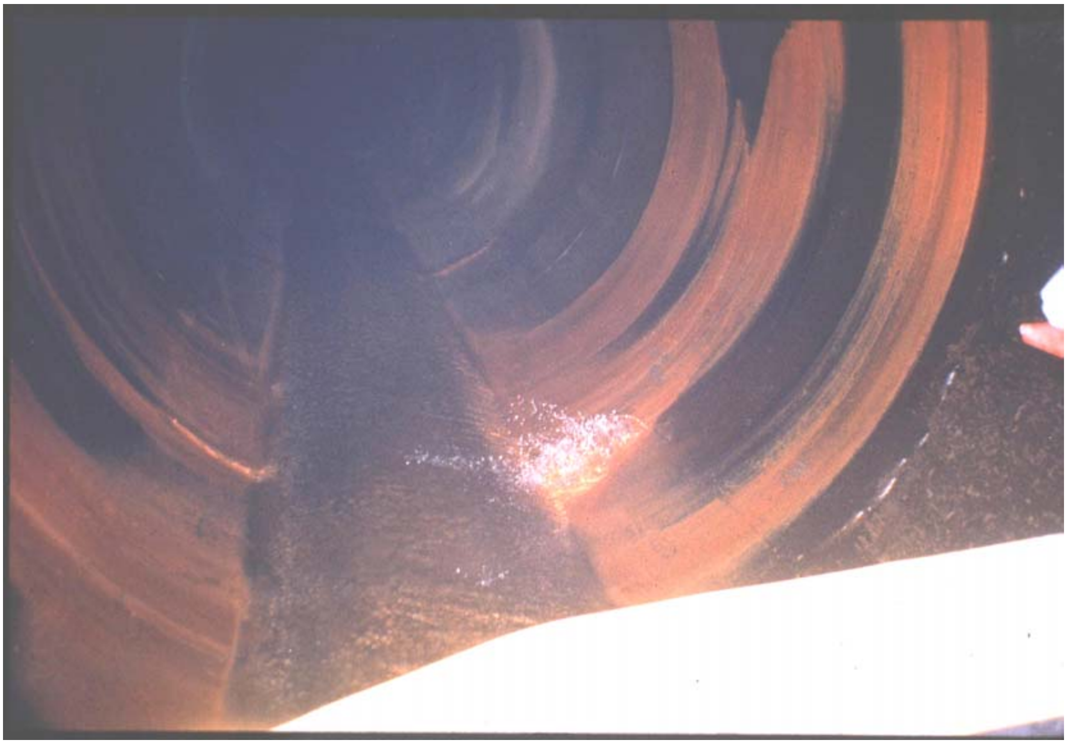

Figure 8. Water Inflow from Check Valve.

Figure 9. Completion of Inspection on the Henry J

ROBERTS TUNNEL REFERENCES

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MOFFAT TUNNEL

Follow up repair work and upkeep are needed to maintain the tunnel facility in its existing excellent operating condition.

PROJECT

The project consists of sealing cracks in the concrete-lined section of the Moffat Water Tunnel from the West Portal eastward along the tunnel alignment to the water diversion shaft for a distance of approximately 2700 feet. (tunnel stations 2723+61 to 2696+62)

PURPOSE

The purpose of the project was to seal the circumferential shrinkage cracks in the concrete lining of the western portion of the Moffat Water Tunnel in the area located from the West Portal eastward to the water diversion shaft, a distance of approximately 2700 feet, in order to stop or greatly inhibit water flow from the Moffat Water Tunnel into the Moffat Railroad Tunnel.

FACILITIES LOCATION

The Moffat Water Tunnel was originally the pilot bore for the famous Moffat Railroad Tunnel and was constructed to convey water from the Williams Fork and Moffat collection systems under the continental divide into South Boulder Creek to Gross Reservoir. Gross Reservoir stores and regulates western slope water coming from the Moffat Tunnel through South Boulder Creek before it goes on to the South Boulder Creek diversion to Ralston Reservoir and the Moffat Water Treatment Plant.

The Moffat Tunnel is 6.3 miles in length, designed to be operated under pressure and is lined with concrete to a finished diameter of 10'6", except for the western most section of 2,700 feet that is 5 feet 8 inches in diameter (Figure 10). At capacity, the tunnel can transport 827 million gallons of water per day or 1,280 cfs (cubic feet per second). The West Portal elevation is 9,091 feet above sea level, the tunnel apex elevation is 9,244 feet above sea level (located approximately 4 miles from the West Portal) and the East Portal elevation is 9,205 feet above sea level. The tunnel essentially operates as a pressure tunnel from the West Portal to the apex and then as a gravity tunnel from the apex to the East Portal. The pilot bore was completed in 1927, the tunnel was enlarged and partially lined with concrete in 1935-1936, the concrete tunnel lining was completed in 1958 (Figure 11). The tunnel was mined by conventional methods (drill and blast), supported in fractured zones and fault zones by steel support ribs or wooden ribs and wooden lagging and then fully lined with concrete and reinforced concrete. The concrete lining throughout the tunnel is thicker than the design designated. Backfill grouting was accomplished throughout the tunnel to insure complete and uniform transfer of the rock loading to the lining.

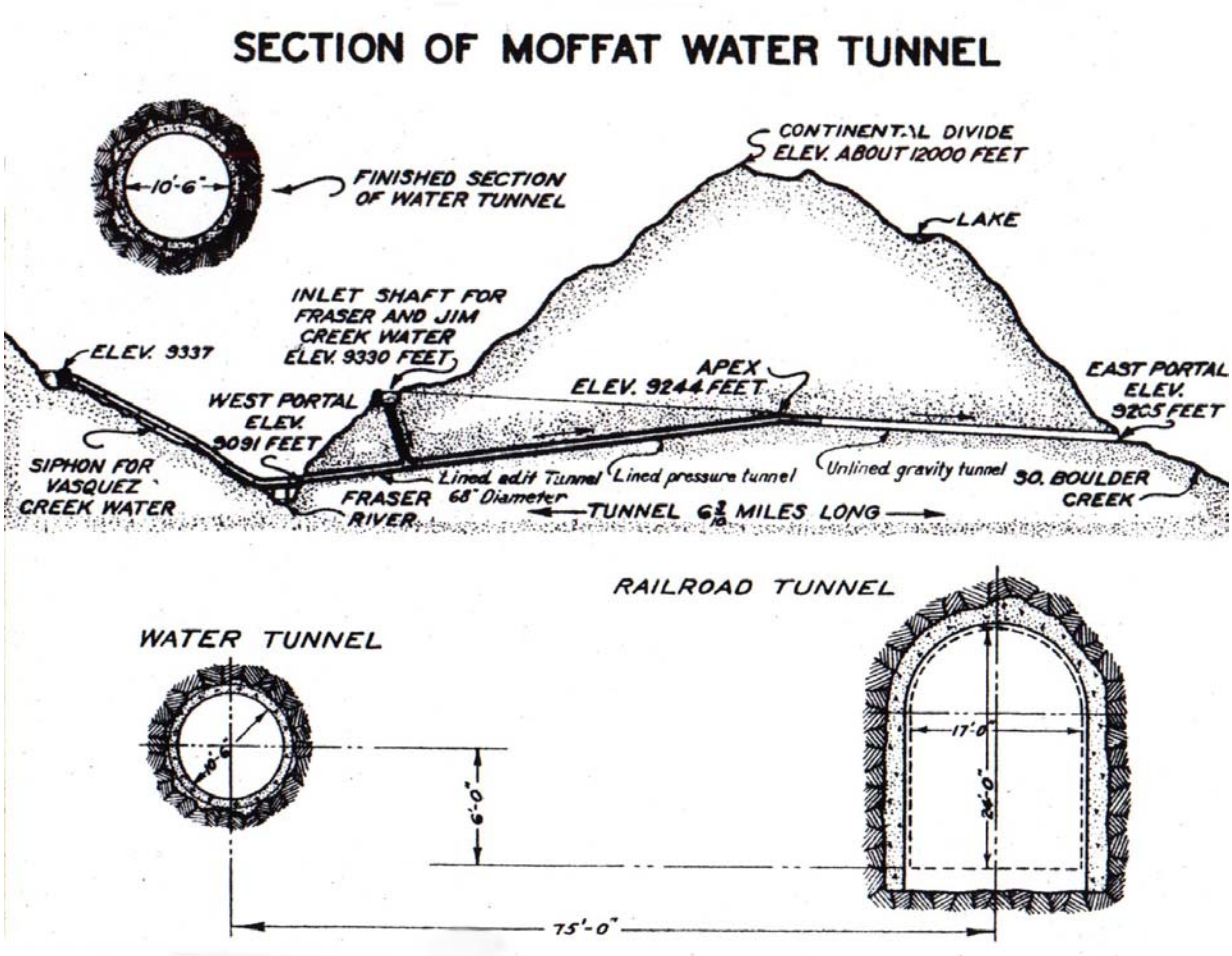

Figure 10. Profile and Section of the Moffat Tunnel.

Figure 11. Placing Steel Reinforcement for Tunnel Lining.

PROJECT SITE GEOLOGY

Bedrock Geology. The Moffat Tunnel transects metasedimentary and igneous rocks of Precambrian age. These rocks are predominantly biotite gneiss in the western half of the tunnel but locally contain layers and pods of biotite-quartz-plagioclase schist, sillimanite and hornblend-gneiss. The rocks penetrated by the tunnel are predominantly migmatite consisting of interlayered biotite-quartz-plagioclase gneiss, granodiorite and quartz monzonite and massive to strongly foliated fine to coarse-grained quartz monzonite and granodiorite. The rock intersected by the tunnel is also jointed, folded and contains numerous faults of various displacements. The faults trend predominantly in a northeasterly direction. The most dominant fracture and fault zones occur in the western one mile of the tunnel and are most probably associated with the Berthoud Pass Fault Zone.

Geology and Tunnel Inspection. The mapped surface geology and tunnel geology information, as well as knowledge of the conditions in the tunnel during construction are valuable tools for the evaluation of conditions in the tunnel lining during inspection and any recommended potential maintenance procedures.

Knowledge of the geology including rock type and strength, fracture systems, locations of faults and groundwater inflow zones offer valuable information to predict problem areas, as well as, aid in solutions for maintenance procedures if necessary.

The possibility that fault movement could damage the tunnel always exists since numerous faults and shear zones occur along the tunnel route. However, the probability that significant damage would occur is very low, and the probability that movement to the extent that it would cut the tunnel completely is extremely low. The geologic record of the Front Range of Colorado shows that fault movement has occurred within the past 20 million years, but recent studies by Denver Water and others for the proposed Two Forks project show that no fault movement has occurred for the past 250,000 years and probably not within the past 500,000 years or longer. The tectonic regime prevailing at the present time in the Front Range is such that it is not likely to produce fault movement that would damage the tunnel within the near future, probably much longer than the expected operational life of the tunnel.

The most fractured section of rock occurs in the western one mile of the tunnel, this section is predominantly biotite-gneiss and migmatite and is also most probably characterized by the extension of the Berthoud Pass Fault Zone.

Zones in the tunnel that are potentially susceptible to problems are associated with the fractures, faulting, and alteration zones scattered along the tunnel alignment, particularly in the western 1 mile that is thought to be associated with the extension of the Berthoud Pass Fault Zone.

PROJECT BACKGROUND AND HISTORY

The Moffat Tunnel is a vital operational feature of the Denver Water Department system providing approximately one-third of the water supply to the Denver metro area. The Moffat Water Tunnel was excavated as the pilot bore for the Moffat Railroad Tunnel in the time period 1923-1927. The water tunnel in the section of interest for this project (west portal to the shaft) was lined with reinforced concrete in the time period 1935-1936 and entered into operation in 1937. Generally, the current condition of the concrete lining in this section of the water tunnel is excellent. Maintenance of the Moffat Water Tunnel is the responsibility of the Denver Water Department as per agreement with the Moffat tunnel Commission.

During construction, shrinkage cracks formed in the lining of the Moffat Water tunnel as a natural result of the curing process of the concrete. When the Moffat Water Tunnel is pressurized and in service, water migrates through these shrinkage cracks and into the surrounding rock formations. Some of the water exits into the Moffat Railroad Tunnel that is located 75 feet north of the Moffat Water Tunnel. This water follows the path of least resistance from the Moffat Water Tunnel along fracture systems in the rock, fault zones and construction crosscuts between the two tunnels. Through time, these water pathways have most probably enlarged, washed out or opened up.

In late November of 1992, a larger amount of water than previously observed was flowing into the Moffat Railroad Tunnel at an area located approximately 1390 feet from the West Portal. This water flow stopped as a direct result of draining the water from the Moffat Water Tunnel and taking it out of service in December 1992. During the course of any given year, water continually migrates into several locations of the Moffat Railroad Tunnel from a variety of sources, including the operating Moffat Water Tunnel, ground water and run-off water. These water flows have been routed into the Moffat Railroad Tunnel drain system through the years and not considered a problem. However, the concentration of a large flow of water into one area presents the possibility to threaten the integrity or operations capability of the Moffat Railroad Tunnel.

CRACK REPAIR SOLUTION EVALUATION

Two general methods were considered to reduce the amount of water migrating from the Moffat Water Tunnel into the Moffat Railroad Tunnel as a logical first step.

1) Grout the rock between the two tunnels.

2) Patch the circumferential fractures in the Moffat Water Tunnel from the inside with a system of flexible epoxy and mortar products.

The most logical solution seemed to be to patch the circumferential fractures with a system of flexible epoxy products from the inside of the tunnel. It was decided to patch all the cracks in the 5′8″ diameter section of tunnel located from the West Portal to the water diversion shaft. This scheme eliminated the possibility of the water migrating out of any unpatched cracks and following the same route to the Moffat Railroad Tunnel.

A grouting operation seemed to be more of a gamble and could result in sealing of the drain system in both the Moffat Water and Moffat Railroad Tunnels or migrating into the Moffat Railroad Tunnel along the path of least resistance, thus not sealing the rock between the tunnels completely.

Dr. Don U. Deere, an independent international consultant on dams, tunnels and underground power plants was called upon to visit the tunnel and provide his expertise in providing a solution for the situation. After his inspection of the Moffat Tunnel, Dr. Deere agreed with method #2 and the crack repair work commenced.

CRACK REPAIR CONSTRUCTION

The work for the crack repair project consisted of sealing the cracks in the concrete lined portion of the Moffat Water Tunnel from the West Portal eastward along the tunnel alignment to the shaft for a distance of approximately 2700 feet. The sealing of the shrinkage cracks in the concrete lining of the western portion of the Moffat Water Tunnel was a logical first step to resolve the problem of water migrating out of the water tunnel and into the railroad tunnel.

Crack Sealing in Concrete Lining
Moffat Water Tunnel
Tunnel Station (2723+61 to 2696+62)
Specification No. 93-7
Contractor: Harrison Western Construction Corporation
Subcontractor: Kleen Kut Service

Job Site Tour: March 4, 1993
Contract Amount: $115,228.00
Duration: March 20, 1993 to April 21, 1993
Project Manager/Project Geologist: Susan Steele Weir
Project Inspector: Jim Warden

The work for this project was for sealing of one hundred sixty-five (165) circumferential shrinkage cracks in the western 2700 feet of the Moffat Water Tunnel. The work was divided into two tasks:

1) Preparation of the concrete tunnel lining in the vicinity of each shrinkage crack for application of sealant by means of cleaning, sawing and chipping (routing). There are approximately 165 circumferential shrinkage cracks that range in length from 18 feet to 20 feet. The total is approximately 3100 linear feet of crack to be prepared and sealed.

2) Sealing of each circumferential shrinkage crack.

Methodology. The surface of the exposed concrete needed to be structurally sound and free from contaminates such as oil, grease, dust, dirt and concrete debris prior to the application of any sealant products including water plug, epoxy and mortar. To accomplish this, a thorough cleaning of the concrete surface, saw cut areas and chipped areas was accomplished by wire brush or mechanical wire steel.

The concrete lining could be damp but needed to be free from standing or flowing water for application of sealant products. Area where there was actual flow of water out of cracks was sealed with water plug, lead wool or equivalent prior to application of other epoxy sealants or mortars. Concrete chips, dust or other debris was collected, contained and hauled out of the tunnel and away from the site for disposal.

Preparation of Tunnel Concrete Lining. The concrete lining containing each circumferential crack was prepared by sawing and/or chipping and cleaning to provide a seat for application of the sealant materials. Generally, the two types of cracks to be repaired were: 1) cracks in the concrete tunnel lining and 2) cracks in the tunnel lining that contained old mortar from previous patch applications. Some cracks were a combination of 1) and 2). The methods for the preparation of the concrete lining at each crack are described in a. and b. below (Figure 12).

a. Cracks in the Concrete Tunnel Lining.
Each crack was cut to form a notch with the dimensions between 1 inch and ½ inch in depth and between ¼ inch and ½ inch in width. This cut in the concrete was accomplished by means of a dry mechanical router and concrete saw (Figure 14).

b. Cracks in the Concrete Tunnel Lining that Contain Old Mortar from Previous Patch Applications.
Saw cuts in the concrete were made outside the old mortar patch to a minimum depth between ½ inch and 1 inch on both sides of the patch. All the old mortar and concrete patch material was be removed by chipping to expose the circumferential crack. This crack was cut to a notch with dimensions between 1 inch and ½ inch deep and between ¼ inch and ½ inch wide. This notch is the same as for a. above. These cuts and chipping in the concrete were accomplished by means of a dry mechanical router or concrete saw.

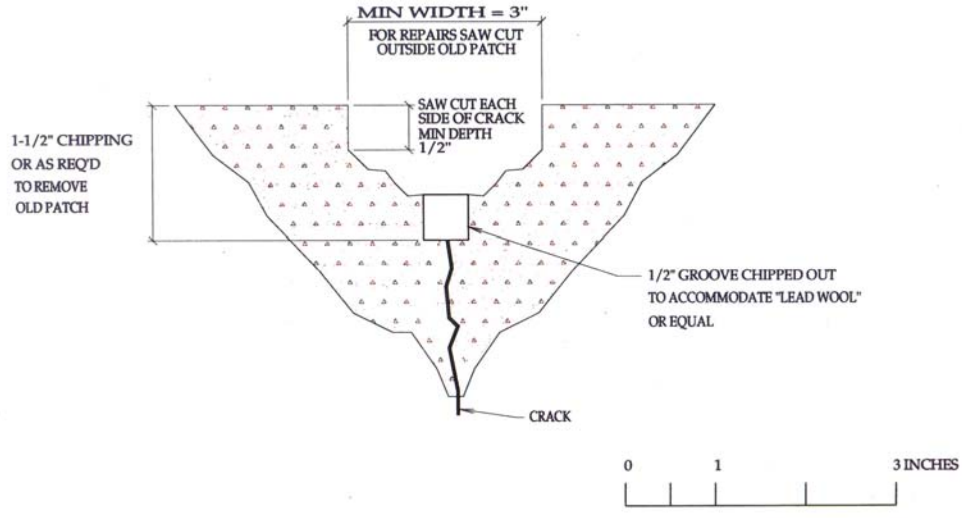

Figure 12. Schematic of Preparation of Concrete Lining at Crack

Figure 13. Saw Cutting Concrete at Crack Location.

Application of Sealant to Cracks In Tunnel Concrete Lining. The following steps were utilized for application of sealant to the cracks in the concrete tunnel lining.

1) Preparation of the concrete tunnel lining was completed prior to application of sealant products including appropriate sawing, routing, chipping removal of old mortar patches and cleaning as outlined in a. and b. above.

2) The structurally sound surfaces of concrete were cleaned free from such contaminants as oil, grease, dirt and debris. This was essential for a bond between the concrete and epoxy sealant products.

3) A system of four products, SIKADUR-51-NS, SIKA SET PLUG, SIKADUR-35-HI MOD LV, and SIKA TOP 123 or their equivalents were then utilized to seal the circumferential shrinkage cracks located in the project area of the western portion of the Moffat Water Tunnel.

   The SIKADUR-51-NS is an epoxy resin adhesive sealant. SIKA SET PLUG is an instantly setting, Portland cement water-stop. SIKADUR 35 HI-MOD LV is an epoxy resin adhesive for sealing absorptive surfaces. SIKA TOP 123 is a polymer-modified Portland cement mortar for patching vertical or overhead surfaces.

   A. The SIKADUR-51-NS was applied to each crack "notch" by means of a caulking gun. The SIKADUR-51-NS was carefully injected only into the "notch" of the concrete and not onto the surrounding areas of concrete.

   B. In zones of actual water flow from the concrete crack "notch", SIKA SET PLUG was inserted to stop water flow according to applications methods outlined. If SIKA SET PLUG was inserted into areas of crack "notches", then after the SIKA SET PLUG set up, a saw cut notch was cut in the top portion and SIKADUR-51-NS was applied according to (A) above.

C. SIKADUR 35-HI-MOD LV was painted over the concrete "notch" containing SIKADUR-51-NS and SIKA SET PLUG in a thin coat that was approximately 2-3 inches wide over the surrounding <u>clean</u> concrete.

D. SIKA TOP 123 was applied on top of concrete "notch" containing IKADUR-51-NS, SIKA SET PLUG and SIKADUR-35-HI MOD LV in areas that a depression had been created in the concrete tunnel lining as a result of removal (chipping) of old mortar patches. The completed sealed and patched areas of concrete tunnel lining formed a smooth level surface consistent with the original diameter of the concrete tunnel lining.

Figure 14. Application of SIKA TOP 123.

CONCLUSIONS

1) Upon completion of the crack repair project, the Moffat Water Tunnel was filled with water and resumed service. An inspection of the Moffat Railroad Tunnel revealed diminished to no water seepage into the area of concern located 1,390 feet eastward from the West Portal of the Moffat Railroad Tunnel or any of the 2,700 feet of tunnel corresponding to the area in the Moffat Water Tunnel sealed during the crack repair project.

2) Subsequent inspections of the Moffat Water Tunnel over the next several years revealed no deterioration to the crack patches in the concrete lining, as well as no increased seepage in the initial areas of concern.

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APPENDIX I – INSPECTION OF THE HAROLD D. ROBERTS TUNNEL

INSPECTION OF THE HAROLD D. ROBERTS TUNNEL - MAY 12, 1992

INSPECTION TEAM

Gene Bode, District Foreman South Platte Area
Susan Steele Weir, Senior Geologist
Bob Schroeder, Superintendent Source of Supply
Martin Rowe, Foreman Light Equipment Shop
Joe Nugent, Dept. Mined Land Reclamation – State of Colorado

PURPOSE OF INSPECTION

The inspection of the Roberts Tunnel completed on May 12, 1992 was the first inspection of the entire length of the tunnel since 1966. The purpose for the inspection was to ascertain the condition of the concrete lining for operational integrity purposes and provide any maintenance recommendations

GENERAL INSPECTION INFORMATION

1) JCI Tunnel Vehicle (Henry J) introduced.

2) Voice communication with inspection team by 2-way radio at Montezuma Shaft only.

3) Pistol shot signaling utilized as communication through the tunnel.

4) Survey odometer wheel rigged to rear wheel of Henry J. to coordinate distance traveled along tunnel with brass plate ¼ -mile markers located at tunnel springline. Utilizing this method, tunnel locations were established for any potential maintenance functions.

5) Trailer pulled by Henry J. containing rescue and operational supplies, 5 hour tank breathing air supply for each inspection team member, ½ hour connecting breathing air masks, inflatable rubber rafts and oars, maintenance tools and supplies, dry change of clothes for inspection team, drinking water and lunch.

6) Utilized tunnel diggers (rain gear slicker pants and jacket) and thigh-high steel toed rubber muck boots as inspection attire.

INSPECTION OBSERVATIONS

1) Concrete lining excellent condition.

2) No noticeable deterioration of concrete tunnel lining.

3) Regularly spaced circumferential fractures with approximately 15’-20’ spacing (result of concrete curing process at construction).

4) Minor additional fractures. All fractures are tight and are not offset or displaced.

5) Some fractures and construction joints are leaching calcium carbonate particularly in zones located between 6.2 and 14 miles from the west portal.

6) Construction joints in the concrete are predominantly tight.

7) Cold joints in concrete are tight.

8) No breakouts in the concrete were observed (a breakout is defined as an area of concrete lining that is missing, thereby exposing bedrock behind the lining).

9) No spalling concrete was observed.

10) Minor erosion of concrete observed in the invert (floor of the tunnel). This condition is assumed to be fairly common in the invert throughout the tunnel due to water flow action.

11) Holes in the concrete lining that were utilized for the back-fill grouting operation during construction were plugged – no deterioration noted.

12) Check valves were functioning properly.

13) Water flow into tunnel from check valves (approximately 5 gpm to 15 gpm) and from some fractures (approximately 1 to 3 gpm).

14) Cement mortar lining section at east portal intact – minor cracking observed in cement mortar lining.

15) Approximately 18-24 inches of flowing water in the invert. This flow is accumulated from a combination of 3 sources 1) flow into tunnel from fractures, 2) flow from check valves, and 3) the slide gate seal at the west portal (access shaft).

16) No gases or bad air were detected during inspection.

17) Oxygen at 19.5% throughout the tunnel; noticeable air current in tunnel.

18) Water temperatures recorded during the inspection were all approximately 50°, except at mile 14.2 where the temperature was 76° (hotspring).

19) The tunnel inspection was completed in approximately 6 hours.

RECOMMENDATIONS

1) Establish a 5-year Inspection Schedule, more frequent inspections if construction projects warrant it.

2) Develop a more effective method of communicating between the tunnel inspection team and personnel at the east and west portals.

INSPECTION OF THE HAROLD D. ROBERTS TUNNEL - APRIL 2, 1996

INSPECTION TEAM

Ed Christensen, District Foreman South Platte Area
Susan Steele Weir, Senior Geologist
Dan Nyman, Caretaker Dillon Dam
Deb Rodgers, Light Equipment Shop
Jim Kelley, Safety

PURPOSE OF INSPECTION

The purpose of the inspection of the Roberts tunnel completed on April 2, 1996 was to insure the operational integrity of the tunnel, as well as, inspect the tunnel after the completion of a construction project in the Montezuma Shaft that involved installation of a pump and waterline in order to supply water for snow making operations at the Keystone Ski Area. The inspection was a double check to insure that there was no construction debris in the tunnel.

GENERAL INSPECTION INFORMATION

1) JCI Tunnel Vehicle (Henry J) utilized for inspection.

2) Microwave radios system utilized for communication between tunnel inspection team and east portal. System had two microwave radios; one mounted on Henry J tunnel vehicle and the second one at the east portal. Communication from the dog-leg in the tunnel to the west portal (approximately 14 miles) was great.

3) Utilized odometer mounted inside the Henry J. to coordinate distance traveled in the tunnel with brass ¼-mile markers located at the tunnel springline. Utilizing this method, tunnel locations were established for any potential maintenance functions. The handheld odometer proved more functional with the direct mileage readout than the survey wheel odometer method utilized in 1992.

4) Utilized magnet grate that was specially designed to match the curve of the tunnel concrete invert and drag behind the Henry J to pick up stray metal objects in the tunnel.

5) Trailer pulled by Henry J. containing rescue and operational supplies, 5 hour tank breathing air supply for each inspection team member, ½ hour connecting breathing air masks, inflatable rubber rafts and oars, maintenance tools and supplies, dry change of clothes for inspection team, drinking water and lunch.

6) Utilized neoprene chest waders and boots for inspection attire; they are much warmer more flexible, and completely waterproof. A great improvement from 1992!

INSPECTION OBSERVATIONS

1) Tunnel conditions were very similar to 1992.

2) Concrete lining excellent condition.

3) No noticeable deterioration of concrete tunnel lining.

4) Regularly spaced circumferential fractures with approximately 15’-20’ spacing (result of concrete curing process at construction)

5) Minor additional fractures. All fractures are tight and are not offset, displaced, or eroded.

6) Some fractures and construction joints are leaching calcium carbonate particularly in the zones located between 6.2 and 14 miles from the west portal.

7) Construction joints in the concrete are predominantly tight.

8) Cold joints in concrete are tight.

9) No breakouts in the concrete were observed (a breakout is defined as an area of concrete lining that is missing, there by exposing bedrock behind the lining).

10) No spalling concrete was observed.

11) Minor erosion of concrete observed in the invert (floor of the tunnel). This condition is assumed to be common in the invert throughout the tunnel due to water flow action.

12) Holes in the concrete lining that were utilized for the backfill grouting operations during construction were plugged – no deterioration noted.

13) Check valves were functioning properly.

14) Water flow into tunnel from check valves (approximately 5-15 gpm) and from some fractures (approximately 1-3 gpm).

15) Cement mortar lining section at the east portal was fairly intact – extensive cracking was observed in portions of cement mortar lining and some sections of lining were missing completely exposing sections of steel tunnel liner.

16) Approximately 18-24 inches of flowing water in invert: this flow is accumulated from a combination of 3 sources 1) flow into tunnel from fractures, 2) flow from check valves, and 3) the slide gate seal at the west portal (access shaft).

17) No gases or bad air were detected during inspection.

18) Oxygen readings at 19.5% throughout tunnel; noticeable air current in tunnel.

19) Water temperatures recorded during the inspection were all approximately 50°, except at mile marker 14.2 where the water temperature is about 76° (hotspring).

20) No debris from the Montezuma Shaft pump and waterline installation construction project was found in the tunnel invert. No damage occurred to the concrete tunnel lining as a result of this construction project.

21) The tunnel inspection was completed in approximately 6 hours.

RECOMMENDATIONS

1) Continue 5-year Inspection Schedule; more frequent inspections if construction projects warrant it.

2) Communication from the tunnel inspection team and the east portal utilizing the microwave radio was excellent. Recommend acquiring an additional microwave radio for the west portal so continuous communication with the inspection team in the tunnel is possible.

3) Remove and repair or replace damaged cement mortar lining, in the section of tunnel near the east portal.

INSPECTION OF THE HAROLD D. ROBERTS TUNNEL - SEPTEMBER 21, 1999

INSPECTION TEAM

Ed Christensen, District Foreman South Platte Area
Susan Steele Weir, Senior Geologist
Kevin Keefe, Superintendent Source of Suppply
Mike Wilkinson, Foreman Light Equipment Shop
Jim Kelley, Safety

PURPOSE OF INSPECTION

The purpose of the September 21, 1999 inspection of the Roberts Tunnel was three-fold 1) routine observation of the conditions of the concrete tunnel lining in order to address any maintenance needs and, 2) investigation after the installation of the new slide gate in the emergency access shaft at the west portal at Dillon Reservoir and 3) provide additional information for the sediment migration situation through the tunnel.

GENERAL INSPECTION INFORMATION

1) Utilized JCI Tunnel Vehicle (Henry J) for inspection.

2) Utilized odometer mounted inside the Henry J. to coordinate distance traveled in the tunnel with brass ¼-mile markers located at the tunnel springline. Utilizing this method, tunnel locations were established for any potential maintenance functions. The handheld odometer proved more functional with the direct mileage readout than the survey wheel odometer method utilized in 1992.

3) Utilized magnet grate that was specially designed to match the curve of the tunnel concrete invert and drag behind the Henry J to pick up stray metal objects in the tunnel.

4) Trailer pulled by Henry J. containing rescue and operational supplies, 5 hour tank breathing air supply for each inspection team member, ½ hour connecting breathing air masks, inflatable robber rafts and oars, maintenance tools and supplies, dry change of clothes for inspection team, drinking water and lunch.

5) Microwave communications system fully utilized throughout the tunnel. Excellent verbal communications between the tunnel inspection team and personnel at the west portal and east portal.

INSPECTION OBSERVATIONS

1) Tunnel conditions were very similar to 1992 and 1996.

2) Concrete lining excellent condition.

3) No noticeable deterioration of concrete tunnel lining.

4) Regularly spaced circumferential fractures with approximately 15’-20’ spacing (result of concrete curing process at construction)

5) Minor additional fractures. All fractures are tight and are not offset, displaced, or eroded.

6) Some fractures and construction joints are leaching calcium carbonate particularly in the zones between 6.2 and 14 miles from the west portal.

7) Construction joints in the concrete are predominantly tight.

8) Cold joints in concrete are tight.

9) No breakouts in the concrete were observed (a breakout is defined as an area of concrete lining that is missing, there by exposing bedrock behind the lining).

10) No spalling concrete was observed.

11) Minor erosion of concrete observed in the invert (floor of the tunnel). This condition is assumed to be fairly common in the invert throughout the tunnel due to water flow action.

12) Holes in the concrete lining that were utilized for the back-fill grouting operations during construction were plugged – no deterioration noted.

13) Check valves were functioning properly.

14) Water flow into tunnel from check valves (approximately 5 to 15 gpm) and from some fractures (approximately 1 to 3 gpm).

15) Approximately 6 – 12 inches of flowing water in invert accumulated from both flow into the tunnel from fractures and check valves and minor amounts from the slide gate seal at the west portal (access shaft). A new slide gate was installed at the emergency access shaft (West Portal) in spring 1999.

16) No gases or bad air were detected during the inspection in the tunnel from the tunnel. Higher levels of CO was detected from the JCI emissions occasionally due to the lower water level in the invert and more emissions into the air from the JCI tunnel vehicle.

17) Oxygen at 19.5% throughout the tunnel; noticeable air current in tunnel.

18) Water temperatures recorded during the inspection were all approximately 50°, except at mile 14.2 where the temperature was about 76° (hotspring).

19) The tunnel inspection was completed in approximately 5 hours. There was less water in the tunnel invert as a result of the installation of a new slide gate at the emergency access shaft (west portal). With less water to motor through, the Henry J was able to move at a faster velocity for the inspection.

20) The tunnel concrete lining is in excellent condition. The most likely source for the sediments captured in the stilling basin and migrating through the tunnel is the reservoir. See attached memos, Weir to Schroeder – sediment migration tunnel for the complete evaluation.

21) The cement mortar lining section at the east portal area is in very poor condition. The cement mortar lining is more cracked and additional mortar lining has fallen out since the 1996 inspection. The steel lining is exposed in several locations, particularly in the crown of the tunnel.

RECOMMENDATIONS

1) Continue 5-year Inspection Schedule; more frequent inspections if construction projects warrant it.

2) Repair and replacement of the cement mortar lining section at the east portal.
Removal of the damaged cement mortar lining, cleaning of the exposed steel-lined tunnel sections, and application of replacement epoxy coating work began in September 1999. Work was suspended in November 1999 and will be completed in 2000.

3) Adjust emissions on JCI Tunnel vehicle to accommodate less water in tunnel invert.

REHABILITATION OF THE LARAMIE-POUDRE TUNNEL

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Key Terms: tunnel, rehabilitation, tunnel support, underground, cellular concrete, rock support, transbasin diversion, Laramie River, Cache la Poudre River, cave in, expandable pipe, foliation shear zone

ABSTRACT

In May 2000, the center of the 89-year-old Laramie-Poudre irrigation tunnel collapsed, shutting off a much needed water supply. The collapse was investigated, repairs were designed, and construction was carried out during the next ten months to allow water to flow for the 2001 irrigation season. The ground consisted of massive Precambrian granites and gneisses with foliation shear zones. The numerous cross sections and support types in the tunnel required a wide variety of rehabilitation techniques. The caved area was mined out and supported with heavy steel sets and a cushion of cellular concrete grout. Sections with square timber sets were lined with an expandable steel pipe and grouted in. Tunnel sections lined with concrete arches were repaired with shotcrete and grouted to remove voids and provide a solid contact between the ground and the support. Other parts of the tunnel required a combination of rockbolts, mine straps, and shotcrete. Enough of the tunnel was repaired by May 2001 to allow irrigation water to flow. The remainder of the eastern half of the tunnel was repaired by December 2001. The western half of the tunnel had temporary support installed and now awaits permanent repairs.

INTRODUCTION

The 2.1 mi (3.4km) long Laramie-Poudre Tunnel is located about 54 mi (87km) northwest of Fort Collins in northern Colorado, 45 mi (72km) up the narrow Poudre Canyon, as shown in Figure 1. The tunnel was driven from 1909 to 1911 at the narrowest point between the Laramie River in the west and the Cache La Poudre River in the east. The 10 ft wide by 8 ft tall tunnel provides water for agricultural, municipal, and industrial uses along Colorado's northern Front Range.

In May 2000, while irrigation water was flowing, the center of the tunnel collapsed, flooding the upstream area and slowing the water to the Poudre River to a trickle. The extent of the damage was assessed and repairs started that fall. The repairs consisted of re-mining the collapsed section and rehabilitating areas where the support had weakened.

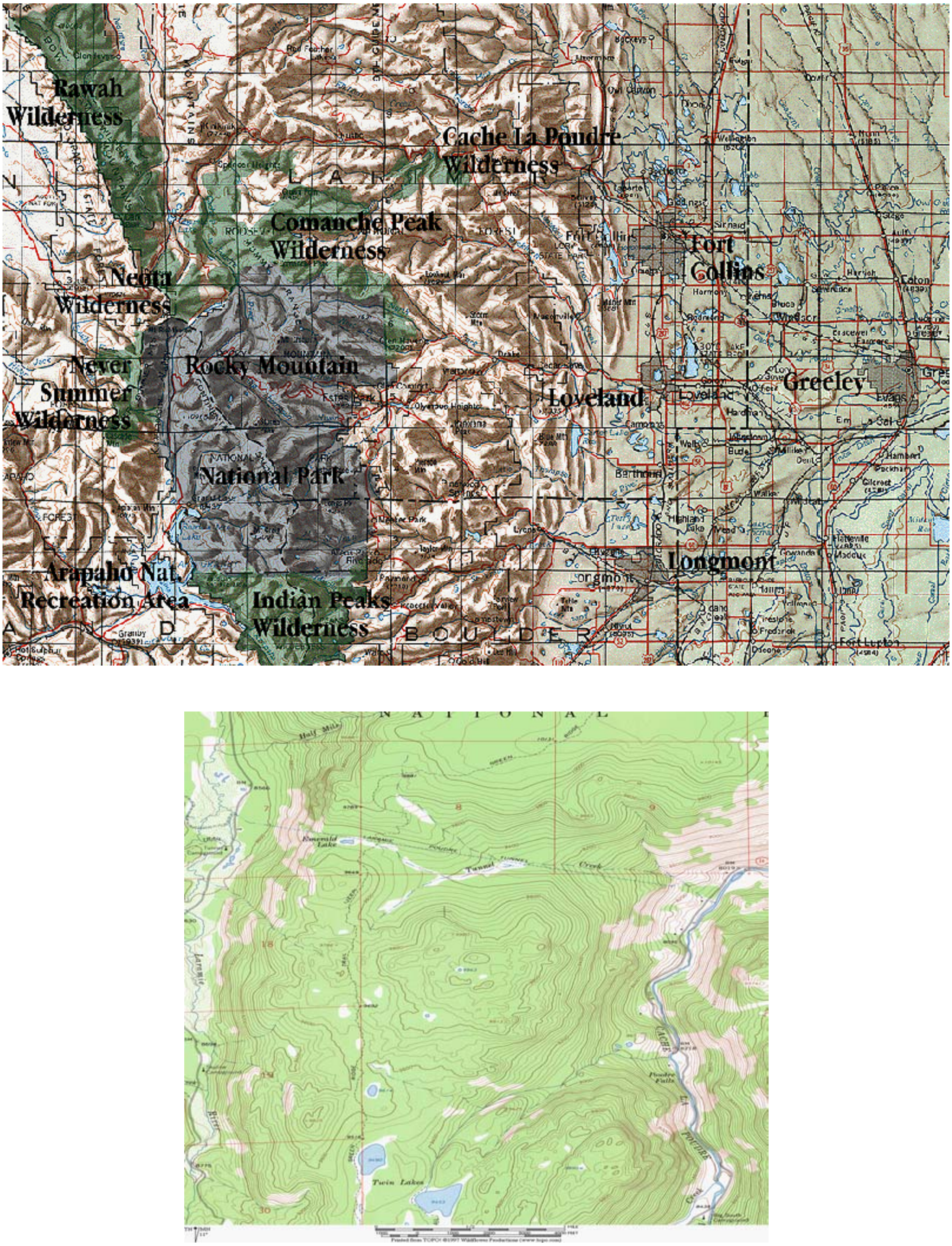

Figure 1. Map of project location.

TUNNEL HISTORY

Construction work on the tunnel started on Christmas Day, 1909. On the east side, a large camp was built. It had an office, black smith, bunkhouse, mess hall, shop, hospital, and power plant (Case 1995). To provide the site with power and compressed air, a dam was built 2 mi (3km) upstream at Poudre Falls. Water was piped to the site in a wood pipeline and used to run a generator and compressor. On the west side, a smaller camp was built and power lines were run to it from the east side.

On the east side, the tunnel was driven with a rough rectangular cross section 10ft (3m) wide and 8ft (2.4m) tall. Leyner No. 7 water drills were initially used, but the hard ground required the development of a more robust drill, which later became the Leyner No. 8 (Brunton 1911). For explosives, a blasting gelatin having 60 percent of the strength of pure nitroglycerine was used for most of the drive. To aid in mucking, steel plates were placed on the invert in front of the face. These provided a smooth surface from which to shovel. Mules were then used to haul the muck cars. While the tunnel portal is hidden, the muck pile is visible from Highway 14 and is shown in Figure 2.

Figure 2. Portal and muck pile on the east side.

On the west side, the tunnel was driven with the same dimensions, but the crown was slightly arched. Tunneling proved to be significantly more challenging on the west side, mostly because it required mining downgrade (1.7 percent). To avoid a massive flood from the Laramie River, the tunnel was driven from a temporary portal approximately 60ft (20m) above the eventual inlet. The temporary adit intersected the tunnel alignment about 200 ft (60m) in. After the remainder of the tunnel was complete, the 200 ft (60m) at the upstream end were driven along the final (lower) alignment to intersect with the ditch and intake structure. The upper adit and portal were then abandoned. Constructing the tunnel through the adit above the tunnel alignment kept the tunnel safe, but the steep decline of 25 percent (Brunton, 1911) created a serious muck-hauling problem. While the tunnel kept clear of the river, groundwater percolating through fissures collected at the face and made work difficult. In October 1910, when advance rates in the east proved to be good enough to make the schedule, work on the west side was stopped.

The tunnel was completed on July 27, 1911 amid great pomp and ceremony. However, court battles between the states of Colorado and Wyoming over water rights prevented water from flowing until 1914 (Case 1995). Since then the tunnel has run water every year, but at a court mandated maximum of 350cfs (9.9 m^3/s) instead of the original design of 800cfs (22.7 m^3/s). Sections of the tunnel were rehabilitated periodically with major operations in the 1940 and early 1970s.

GEOLOGY

The tunnel penetrates a massive complex of Precambrian granites and gneisses, part of the Front Range of the Rocky Mountains. Locally, nodules and layers of strongly altered biotite schist are encountered. The strong granites and gneisses provide excellent ground conditions, allowing over 80 percent of the tunnel to remain unsupported. However, areas with soft, sand like biotite invariably require support.

Some of the areas with the biotite schist could be called “foliation shear zones”, as defined by Dr. Don U. Deere in his 1973 paper “The Foliation Shear Zone-An Adverse Engineering Geologic Feature of Metamorphic Rocks” (Figure 3). These foliation shears are thin, sheared zones along the foliation of metamorphic rocks. The shears occur in the weak layers, typically in mica schist zones. Deere notes that the shears themselves tend to only be a few inches thick, but are surrounded by weakened rock. The zones typically contain a crushed mica schist gouge (including clay minerals) within the shear. The surrounding rock tends to be partially crushed, slickensided, and tends to have chlorite coatings on the joints. In regards to groundwater, the foliation shear zones tend to act as drains for water running parallel and as dams for water running perpendicular to them. Foliation shear zones originate with differential movement caused by folding, stress relief, or thrust faulting. These zones tend to exhibit low shear strength and high compressibility, leading to roof instability in tunnels.

Figure 3. Photograph of foliation shear zone.

Two main joint sets are found throughout the tunnel. The first is near vertical and perpendicular to the tunnel axis. Where encountered individually, these joints just exhibit local overbreak. Where several joints are grouped together, short sections of support are required. In some areas, this joint set is a foliation shear. The second joint set is subparallel to the tunnel and dips at a shallow angle. In numerous places a joint can be seen slowly coming up from below springline, defiantly making its way to the crown. Full support, often for more than 130ft (40m), is required where the joint moves across the crown. Typically these joints are clay filled, but locally they are healed with calcium carbonate. Some of the joints are foliation shears and are filled with mica schist gouge. Areas where the two joint sets intersect typically had very high crowns and required significant support. The caved section featured both joint sets with close spacing along with biotite schist pockets. This was by far the largest foliation shear zone on the project.

Groundwater flow in the tunnel is minimal, ironically making working in an irrigation tunnel dry. During the 2001 rehabilitation, local dripping was observed in some of the more jointed sections. About 1300ft (400m) in from the east portal, a 60ft (20m) long section is covered in calcium carbonate stalactites. Small quantities of water dripped from here and covered old tunnel muck and debris, turning it into a stone like substance. Water inflow was also noticeable near both portals, in the more weathered ground.

INVESTIGATION

Rocky Mountain Consultants Inc. (now Tetra tech RMC) was contacted by the Tunnel Water Company in the summer of 2000 to investigate the collapse, recommend a solution, develop plans and specifications, and provide construction oversight. Engineers and geologists examined the surface above the tunnel and both sides of the tunnel from the portals to the caved area. Near the center of the tunnel, on the west side, a timbered section gave way to a nearly vertical face of collapsed rock (Figure 4). On the east side, debris from the collapse was visible several hundred meters downstream. As one approached the caved area, the debris built up like a ramp until the tunnel became impassable.

Figure 4. Photograph of west (upstream side) of cave in.

As shown in Figure 5, the accessible areas of the tunnel revealed a multitude of support types. The most common support was square timber sets with rock back packing. In some areas the timbers were edge to edge, making a rectangular wooden flume. In other areas timbers had 2-6in (5-15cm) gaps between them. Some of the timbers appeared to be from the original construction, while others were certainly put in later. Another common support type was the concrete arch. The arches appeared to have been formed and filled around existing, probably failing, timbers. Reinforcing steel was found exposed in the concrete in some instances.

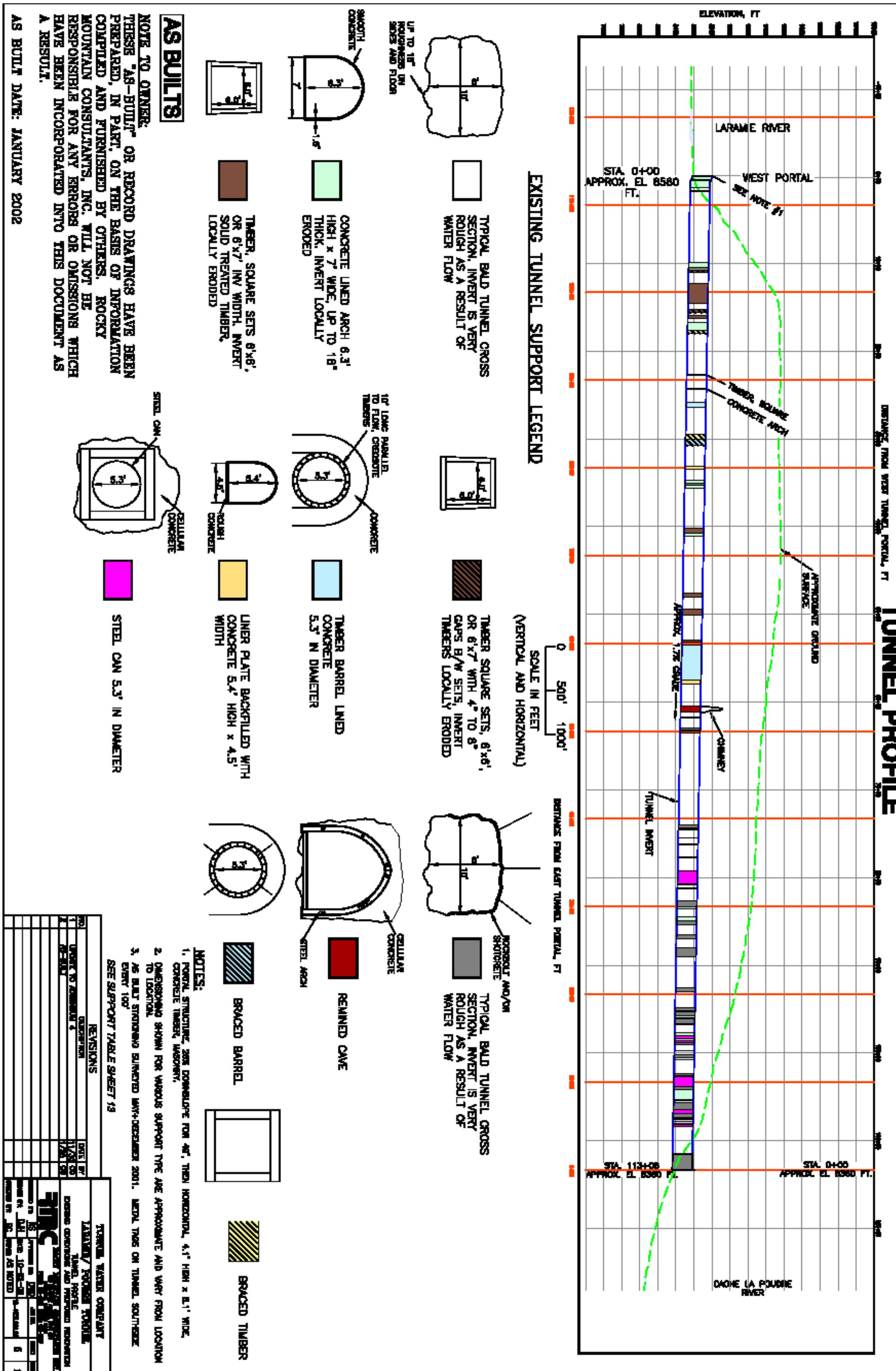

Figure 5. Diagram of various cross sections in the tunnel and profile.

In two areas of the tunnel, liner plate with concrete backfill provided the support. These areas, 4ft by 4.5ft (1.2m by 1.4m) proved to be restrictive to both the engineers and the water. Calculations showed that when the tunnel was running at maximum capacity, the liner plate sections had pressurized flow. This was seen in the field where the upstream side of each section had rubble piled up and the downstream side had holes, eroded in the invert, more than 3ft (1m) deep, 30ft (10m) long and as wide as the tunnel.

The most unusual support was a timber "barrel." This timber barrel consisted of 10ft (3m) long timbers that were laid end-to-end, parallel to the flow, and arranged in a circle like the staves of a barrel. Behind the timber barrel was cast in place concrete. The ends of the barrel sections had attractive transition areas made of stones cast within the concrete. Figure 6 shows a typical section.

Figure 6. Photograph of west side timber barrel section.

DESIGN

Designing the rehabilitation was the next major challenge. To be successful, the following criteria had to be met:

1. Clear the blockage
2. Support potentially weak areas
3. Keep the tunnel capable of flowing its legal volume of 350cfs (9.9 m^3/s)

4. Have the tunnel active for the 2001 irrigation season
5. Keep total costs below $4 million

To achieve these goals, the project was split into three phases. Phase one consisted of remining through the caved section, supporting areas with failing supports, and making permanent repairs to areas in the middle of the tunnel. Phase two consisted of placing permanent support in the entire eastern half of the tunnel. Since access to the east side of the tunnel would be difficult in the future, attention was focused on repairing that side. Phase three, which is scheduled for future work, will consist of replacing interim supports on the west side with permanent structures. Phase three would also likely include a new intake structure on the west side.

MOBILIZATION

When Robison Construction Inc. (RCI) mobilized on site in November of 2000, they had to deal not just with a challenging tunnel repair, but also with major logistical obstacles. The tunnel location, 45 mi (72km) up the narrow canyon and 54 mi (87km) away from nearest city, made equipment procurement and servicing more difficult and time consuming. The actual site access road was a four-wheel drive/hiking trail that had been widened a small amount. The road size and eventual tunnel restrictions severely limited the size of equipment that could be used. The remote location also yielded very limited housing for the crews.

THE CAVE-IN

The critical task in the tunnel was to clear and support the caved-in section. The task was greatly complicated by the size restrictions enroute to the caved area. The narrow timber sets allowed only the smallest equipment through. All mucking was accomplished with Eimco 12B and 22B pneumatic overshot muckers loading 3cy (2.2m^3), muck cars, invoking many fond memories of mining in the 1950s. As the face slowly advanced, W6x25 arched steel sets were stood every 4ft (1.2m) for support. Steel lagging was welded between them. Heavy ship channel was used as crown bars to provide a canopy ahead of the face. The muck was a mixture of sand, clay, gravel, and rock blocks ranging in size from a hat (less than 1ft^3, 0.03m^3) to a large van (450ft^3, 13m^3). During the advance, the old timber sets appeared in the face. They leaned a bit to the left, but were otherwise intact within the field of muck. Figure 7 shows a typical view of the face.

When the crews broke through on March 6, 2001, they looked back on a 1200cy (920m^3) cave, 60ft (20m) long. Above the tunnel, the cave reached up 40ft (12m), but to the north the cave reached at least 80ft (24m) before pinching out in the darkness. A picture taken from the west side shortly after breakthrough appears in Figure 8.

The cave was too large to be filled, given the budget, equipment size, and time constraints. Hence, an 8ft (2.5m) thick cushion of cellular concrete grout was pumped above the fully lagged steel sets to absorb the impact of any future rockfall. The cellular concrete consisted of water,

Figure 7. Photograph of face during mining. Note the crown bars and make up of the muck.

Figure 8. Photograph of final mucking and cave, taken from the west side.

cement, and foam, yielding a very flowable, lightweight material with 500psi (3.4Mpa) compressive strength.

TIMBERED SECTIONS

The same type of timber sets that had failed in the cave had to be rehabilitated in other areas to assure a long lifespan for the tunnel. Some of the shorter sections in relatively good ground were removed and replaced by 6 ft-long (2m) grouted rockbolts, mine straps, and shotcrete. The shotcrete was a dry mix containing steel fibers and silica fume and was applied with a small, easily transportable pump.

The longer timbered sections had to be repaired in place with an expandable steel pipe, placed inside the sets and grouted into place. The pipe was made of 3/8in (1cm) thick steel that was split longitudinally rolled together so that the ends overlapped. This compressed pipe was able to navigate tight spots within the tunnel while being moved into place with the locomotive. It allowed the pipe to fit through tunnel constrictions instead of having to locally remove support and re-mine. Once the pipe was in place, the overlapping joints were expanded to 5.3ft (1.6m), filling the inside of the timbers. After all the joints were welded, the ends were sealed with shotcrete bulkheads. Next, cellular concrete grout was pumped though grout ports in the upper haunches of the pipe. The grout filled both the annular space between the pipe and timber as well as the voids in the crown. 414ft (126m) of tunnel were repaired in this manner. Figures 9 and 10 show the repair in progress and the completed repair respectively.

Figure 9. Photograph of expandable steel pipe with grout hoses in place.

Figure 10. Completed repair of a timbered section.

OTHER REPAIRS

During the last 90 years, various sections of the tunnel had been repaired with a cast in place concrete arch. In most sections this appeared to be over existing timber sets. Most of the concrete arches had visible voids and "punky" areas where the concrete had decayed and now showed the remains of corroded rebar. These concrete arched sections were repaired in three steps. First, steel c-channel arches were bolted into place on 4ft (1.2m) centers. Next the steel arches and concrete (including the invert) were covered with 3in (8cm) of shotcrete. Finally the voids behind the concrete were filled with cellular concrete grout.

Most of the remainder of the tunnel was in good ground and required no further support. Blocky areas were locally bolted. The more sheared areas were supported with a combination of rockbolts, mine straps, and steel fiber reinforced shotcrete. Shotcrete was also used in transition areas between different cross sections to provide a wear surface for the turbulent water.

On the west side of the cave in, repairs were only temporary. Two timbered sections were supported with internal steel braces. These braces had wood lagging between them to reduce friction and allow the designed amount of water to flow. A steel deflection wall was built at the upstream end of one timber section to funnel water directly into the braced sets and keep water from flowing behind the timbers. Timbers had fallen in on another timbered section. These were removed and the roof above was supported with 42 rockbolts and mine straps. One of the "barrel" sections had deformed on the north side. This section was supported by rounded arch steel sets made of half pipes. No repairs were done between the first liner plate section and the cave in because of access restrictions. Future permanent repairs on the west side will likely feature many of the same repair techniques used on the east side.

SUMMARY AND CONCLUSION

Water flowed on May 16, 2001 and ran for the entire irrigation season. That fall the contractor remobilized and finished the permanent repairs on the east side. In contrast to the original builder, all work was done by Christmas of 2001.

The Laramie-Poudre Tunnel rehabilitation was a challenging, interesting, and successful project in an 89 year old tunnel though granite and gneiss with foliation shear zones. It was investigated, designed, bid, and constructed (phase 1) in 10 months. Repairs required the use of both old techniques, such as, crown bars, overshot muckers, bolts, and mine straps, and innovative techniques such as cellular concrete grout and expandable pipe. The east side of the tunnel should be secure for many years to come and the west side should allow water to flow until full repairs can be made.

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WOLF CREEK PASS
UPPER NARROWS PROJECT

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Key Terms: rock mass, volcanic tuff, alpine engineering, explorations, tunnel, rock cuts, ground nail walls

ABSTRACT

This paper presents a geotechnical overview of the Upper Narrows Project, a complicated and high profile upgrade of Highway 160 east of Wolf Creek Pass in the San Juan Mountains of southern Colorado. It presents salient project information of interest to geologists and geotechnical engineers, especially explorations, geologic conditions, geotechnical design, and construction. The project involves the upgrade of one-mi (1.6 km) of highway in a steep alpine canyon at an elevation of 9000 ft (2743 m). Project components include a 906-ft-long (276 m) tunnel, rock cuts up to 150-ft-high (46 m), ground nail walls, mechanical stabilized earth walls, and roadway improvements. Ground conditions consist primarily of Fish Canyon volcanic tuff bedrock overlain by thin discontinuous talus and other colluvial deposits. Rock joint patterns had a controlling influence on project design and construction. Project challenges include the rugged alpine setting, heavy highway traffic, steep canyon, tight work areas, and high environmental and visual standards. Project components utilizing innovative technologies of special interest include the rock cut design, ground nail walls, tunnel support and tunnel lining systems, and emergency provisions for the tunnel.

PROJECT OVERVIEW

The Upper Narrows Project is part of the Wolf Creek Pass East Roadway Reconstruction Project for the upgrade of Highway 160, with the goal of improving the safety and level of service for highway users. It is approximately one-mile long and the first phase of approximately eight mi (13 km) of planned improvements on the east side of the pass. It is seven mi (11 km) below Wolf Creek Pass, six mi (10 km) below the Wolf Creek Pass Ski Area, and 10 mi (16 km) west of the town of South Fork as shown in Figure 1.

The project owner is the Colorado Department of Transportation (CDOT), Region V, with partial funding from the Federal Highways Administration (FHWA). Project design was led by Carter Burgess with the Denver office of Haley & Aldrich, Inc. (now Lyman Henn, Inc.) responsible for geotechnical and tunnel design. Project construction began in the fall of 2000 and is expected to be complete in the spring of 2004.

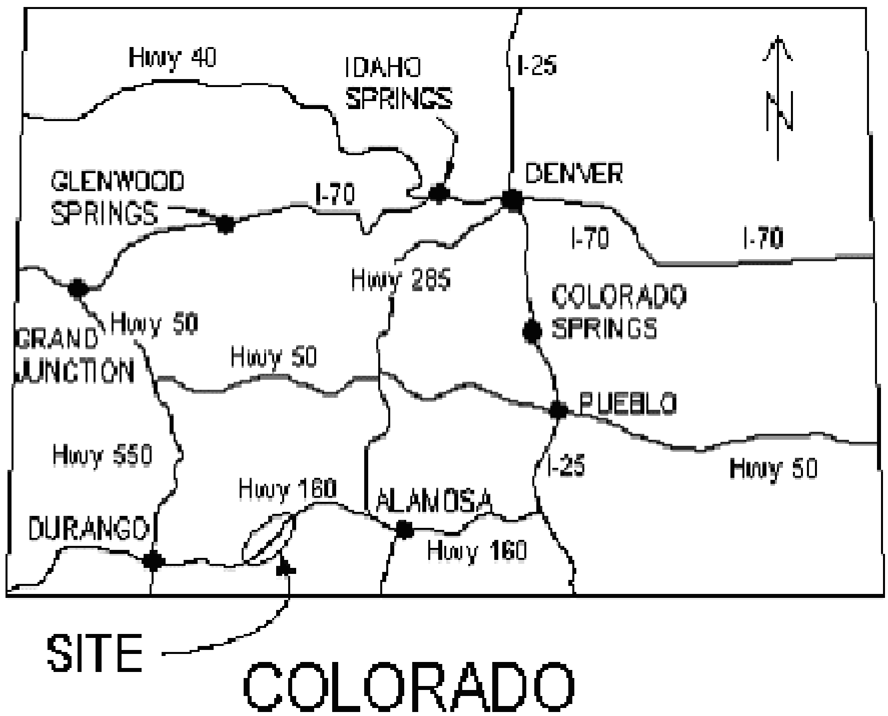

Figure 1. Project Location.

Project Conditions, Goals and Challenges

The project is in a steep canyon at an elevation of 9,000 ft (2743 m) in scenic and harsh alpine environment, and adjacent to a protected stream. Within the project limits US 160 is adjacent to Pass Creek, which is a tributary to the South Fork of the Rio Grande. The highway is perched 150 ft (46 m) above the creek, and with steep talus slopes and cliffs above and below the road. Annual snowfall in the area is between six and eight ft (1.8 and 2.4 m). Key considerations for project design were the remote alpine location, maintenance of heavy highway traffic, high environmental and visual requirements, and the desire by CDOT to minimize maintenance and operational activities.

Due to the scenic setting and location within the Rio Grande National Forest, it was paramount to maintain high environmental and visual standards. There was strong consensus within the design team, CDOT, and the Forest Service that the visual aesthetics and natural appearance of the canyon be maintained to the highest degree practicable. Visual appearance of the project therefore played a significant role in the planning and design process, and landscape architects provided input into the design process.

Prominent wildlife species in the area include elk, deer, raptors, lynx, and trout. The lynx was of special interest as it was recently reintroduced to the area, requiring special highway crossings suitable for these wild cats. Although trout are not an endangered or threatened species, it was necessary to maintain the habitat in relatively undisturbed condition, which effectively disallows disturbance of Pass Creek and the South Fork of the Rio Grande.

Traffic on the highway is a combination of passenger and commercial vehicles. Hazardous cargo will be allowed in the tunnel without special restrictions. Due to heavy use and lack of alternative routes for detours, traffic maintenance during construction was a major consideration.

Two additional project constraints were the limited availability of power and lack of direct communications for tunnel lighting, ventilation, and emergency response. Although a high voltage power line is located a couple miles above the project, the available power is limited. There are presently no communication lines at or near the project that can be easily accessed for the tunnel oversight and safety systems. Construction is unusually challenging because of the alpine setting including the remote location with inconvenient access, short construction season, harsh winter conditions, tight work and staging areas, and the rugged topography.

Project Components

The Upper Narrows project is approximately one-mi-long (1.6 km) with a layout as shown in Figure 2. Highway improvements entail changing from the current two lanes to a combination of two and three lanes, longer radius curves, wider shoulders, and other safety improvements. Features included in the project of special interest to geologists and geotechnical engineers are a 906-ft-long (276 m) tunnel, numerous rock cuts to 150 ft high (46 m), ground nail walls, and mechanical stabilized earth (MSE) walls. Although the project is aligned north/south, Highway 160 is an east/west road and the portals are designated East and West.

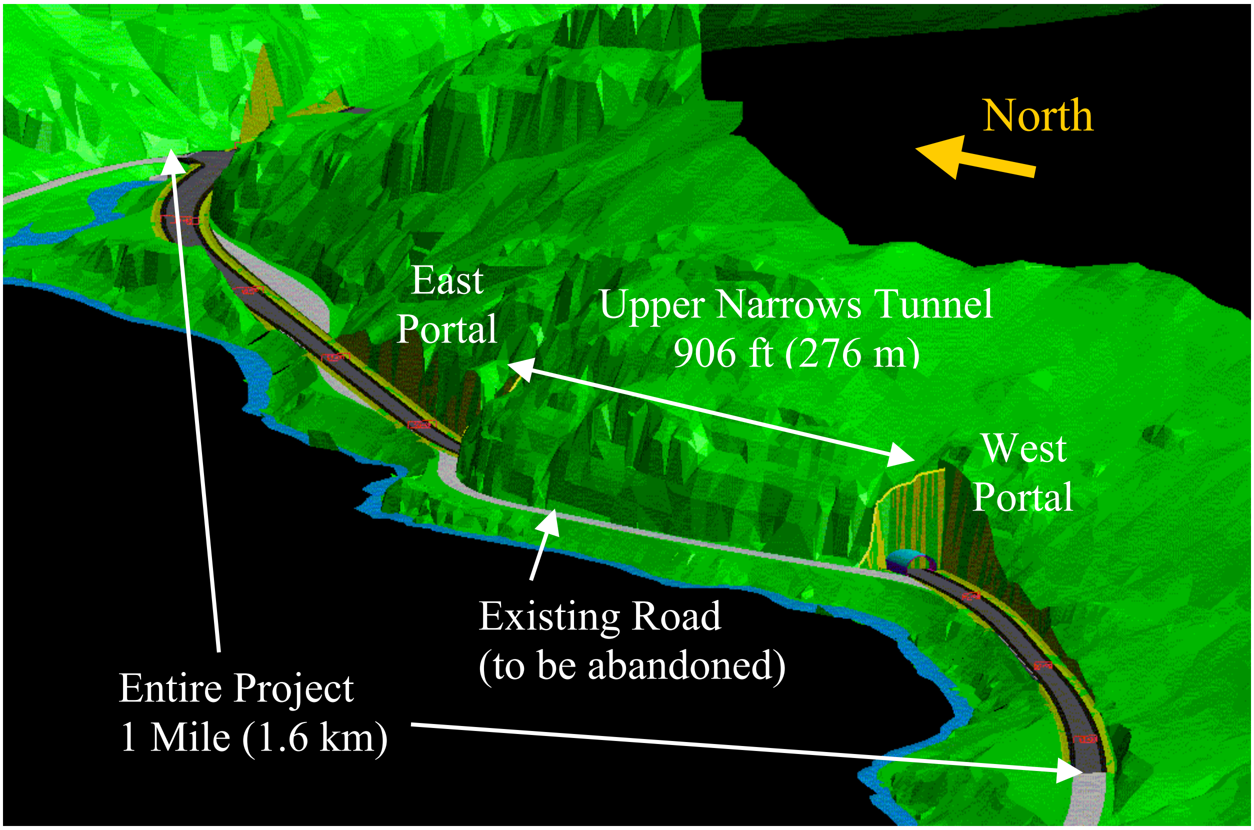

Figure 2. Project Layout.

Construction is in two contracts primarily due to the availability of funds and legal requirements for funding of CDOT projects. Phase One is for exterior rock cuts, and excavation and initial support of the tunnel. The contractor was Kiewit Western Co. and work was completed in January 2003. Phase Two will be for tunnel final lining, safety systems, MSE walls, and roadway improvements. The contractor will be ASI RCC, Inc. and it is scheduled to begin construction in the spring of 2003, and to be completed in 2004.

EXPLORATIONS

Explorations focused on locating the interface between rock and soil, and determining the character and engineering properties of the rock mass and colluvium. Borings, mapping of rock outcrops, seismic geophysics, and laboratory testing provided the necessary data. Most of the borings were core holes in rock, with augers and casing advancers sometimes used through the coarse granular soils. Seismic geophysics lines were used primarily to determine the interface between soil and rock, and to obtain seismic velocities of the rock mass for correlation with rock mass quality. Rock outcrops, including both natural cliffs and preexisting road cuts were mapped for rock mass quality, joint orientations, joint properties, and water seepage. Rock jointing was of special interest, and joints were mapped for orientation, dip, frequency, and engineering properties.

Explorations were conducted in two phases. Phase One was conducted in the fall of 1988 to determine overall conditions and conditions specific for the tunnel. It had a scope of 14 core holes with 1005 ft (306 m) of drilling, two geophysics lines totaling 525 ft (160), mapping of over 150 joints, and laboratory testing. The laboratory program included testing for uniaxial and point load strength, Brazilian splitting, joint shear, sonic velocity, specific gravity, sodium sulfate soundness, and L.A. abrasion, plus petrographic analysis of the rock. Phase Two was conducted in the spring of 2000 to better understand ground conditions for the walls, with a scope of 14 borings totaling approximately 515 ft (157 m). The rock core utilized several different sizes of barrels depending on the rig including NQ-3 (1.775 in, 45 mm I.D.), BDBGM (1.65 in, 42 mm I.D.), and NX (2.125 in, 54 mm I.D.).

Borings were located on the existing road and the canyon slopes above and below the road. Truck mounted drill rigs were used for borings along the existing road with some holes aligned horizontal or angled. Access to borings outside the road was challenging due to the steep canyon topography, coarse talus, and rock outcrops. Additionally, visual standards and environmental impacts prevented constructing temporary roads to access drill sites. Many of the borings were drilled with portable lightweight Winkie drill rigs mobilized with a helicopter, as shown in Figure 3. These drill rigs disassemble into three major parts and can be easily set up on virtually any slope using a small platform constructed by hand.

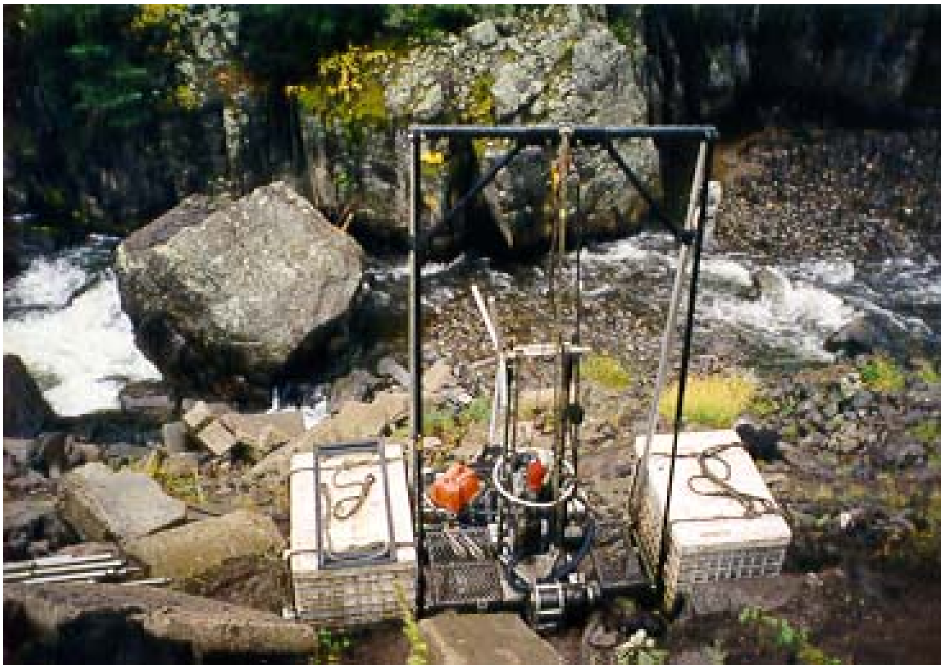

Figure 3. Winkie Drill Rig Above River.

GEOLOGY AND GROUND CONDITIONS

Overview

The canyon for the South Fork of the Rio Grande is steep, over 1500 ft deep (457 m), and dominated by Fish Canyon Volcanic Tuff, which is exposed for the full depth of the canyon. The tuff consists of welded volcanic ash flows occurring in near-horizontal beds typically 20 to 100 ft thick (6 to 30 m). Rock outcrops and cliffs are common in the canyon, forming stair-stepped palisades on the canyon sides. The rock mass is generally competent, fresh to slightly weathered, and massive, with moderately- to widely-spaced joints. The majority of the rock mass is considered to be of fair to excellent quality, but less competent rock exists, both in terms of weathering and rock jointing.

Overlying the bedrock there is a thin veneer of colluvium, predominantly coarse grained talus. The colluvium obscures a complicated rock surface topography including vertical cliffs with a highly irregular profile and angled slopes. In some areas the talus forms small wedges above the bedrock, and in other areas the talus mantles the bedrock with small rock outcrops.

Rock Material Properties

The rock is gray, slightly vesicular welded tuff with lithic fragments from ¼ to 2 in. (6 to 50 mm). It is hard, strong, and abrasive. A summary of material properties based on laboratory testing of intact specimens of the rock from core holes is presented in Table 1.

Table 1. Properties of Intact Rock

Property	Range	Average
Unconfined Compressive Strength	2,000 to 16,000 psi (14 to 110 MPa)	10,000 psi (69 MPa)
Brazilian Tensile Strength	300 to 900 psi (2 to 6 MPa)	700 psi (5 MPa)
Cerchar Abrasivity	2.7 to 3.4	3.2 (highly abrasive)
Specific Gravity	-	2.4

Rock Mass Jointing

Four main joint sets have been identified in the project area: one primary, two secondary, and one random. The terms primary and secondary do not necessarily define prevalence or frequency, but rather geologic sequence. Cliffs, palisades and benches in the canyon are structurally controlled by the jointing patterns.

The primary joint set is parallel to sub-parallel with the bedding surfaces between different ash flows and deposits. On a wide scale the primary jointing is near horizontal to sub-horizontal often with a shallow dip to the west. On a smaller scale the joints have an undulating character with a wavelength on the order of several ft to tens of ft (one to ten m), and localized dips up to 30 degrees. These joints are relatively continuous, and are traceable for tens of ft to hundreds of ft (10 to 100 m). Spacing ranges from 10 to 50 ft (3 to 15 m), with more frequently spaced joints near the bedding contacts. Most primary joint surfaces are tight and unweathered or lightly weathered with a moderately rough texture. However, some primary joints exhibit a thin zone of tight fracturing with weathering and some clay infilling.

Two secondary sets were identified. These are near vertical, and orthogonal to each other and to the primary joint set. Dips of 80 to 90 degrees are common. Spacing typically ranges from 3 to 15 ft (1 to 5 m), with some jointing occurring in narrow swarms as tight as several inches (75 mm). On a project-wide basis the joints have a consistent orientation; however, the orientation varies greatly with location. At the west portal the secondary sets strike nearly parallel and perpendicular to the tunnel, and at the east portal they strike at angles between 15 and 30 degrees (60 to 85 for complementary set) to the tunnel. Secondary joint continuity varies with some joints traceable for the full height of exposed cliffs; other joints extend for only ten ft (3 m). Although individual joints may or may not be continuous, the trends of the major joint sets are relatively consistent. These joints are typically tight to less than ¼ in. (6 mm) open without infilling, and are fresh to slightly weathered with a moderately rough texture. However, a few of the joints have a clay coating or are open by several inches and infilled with soil, especially joints near the ground surface. Occasionally a third set of randomly oriented joints is also present. These joints are infrequent and can have any orientation. The character of these joints is similar to the secondary joints.

Rock Mass Characterization and Behavior for Tunneling

Descriptions of the rock mass utilized several different systems to present a range of descriptions and expected behaviors. These systems utilize rock mass properties of strength, jointing, weathering, and ground water conditions combined with tunnel size and excavation methods as a basis for rock mass classifications and expected ground behavior specific to tunneling. Two of these, the Geomechanics System (RMR) and the Q-System calculate a number representing rock mass quality based rock mass properties.

The rock mass is described as either Common Rock or Poor Rock, comprising 95 and 5 percent of the ground, respectively. Descriptions of each based on common tunneling classification systems are summarized in Table 2.

Table 2. Rock Mass Descriptions and Characterizations

Ground Description & Category	Terzaghi[1]	RQD (%)	Geomechanics System[2] RMR[3]		The Q-System[4] Q[5]	
			Range	Description	Range	Description
Common Rock	*Massive to moderately jointed*	75 to 96	52 to 59	Fair	16 to 24	Good
Poor Rock	*Moderately blocky and seamy to stratified vertical*	24 to 75	23 to 52	Fair to Poor	0.5 to 16	Good to Very Poor

Notes: 1.Terzaghi (1946), Huer (1974), Proctor and White (1988)
2.Bieniawski (1973), Bieniawski (1989)
3.RMR ranges from 0 to 100 in a linear scale
4.Barton, Lien, and Lunde (1974)
5.Q ranges from 0.001 to 1,000 in a quasi-log scale

Ground Water Conditions

The ground water surface is below the tunnel invert, and is assumed to mimic topography in a muted fashion with flows toward Pass Creek. The ground water system at the tunnel is an unconfined aquifer within joints in the bedrock fed by rain and snowmelt from the ground surface.

Observations of cliffs and rock cuts along the existing highway outside the proposed tunnel revealed some seepage from joints. The magnitude of seepage changes seasonally, with the maximum seepage during spring runoff and summer thunderstorm seasons. Some of the joints are damp seeps, but some discharge enough water to drip or on rare occasion to flow. Flowing conditions were observed only from open joints near the ground surface that were visibly traced to the ground surface and a water source. During construction seeps into the tunnel were moderately spaced and more prevalent on the mountain side than the free side. Seepage quantity increased after rain events with an approximate 48 hour lag time.

DESIGN

Tunnel Layout and Geometry

The tunnel is 906-ft-long (276 m) and penetrates a lobe of rock cliffs to avoid tight curves. It parallels the existing road with a gentle S-shape. The existing alignment located outside the lobe will be abandoned after the tunnel is opened. The tunnel will accommodate two 12-ft (3.6 m) travel lanes in a bi-directional configuration with a finished width of 44 ft (13.4 m) which also includes six-ft (1.8 m) shoulders and four-ft (1.3 m) sidewalks with mountable curbs on both sides. The finished width is slightly wider than most two-lane highway tunnels to accommodate bi-directional traffic even in the event of a stalled vehicle, plus sidewalks for safety and emergency egress on both sides. A modified horseshoe shape with a maximum height of 25.7 ft (7.8 m) was chosen to accommodate the vehicular clearance envelopes, provide an economical arched crown, and minimize excavation volume. There is a 51.6 ft (15.7 m) difference in elevation between the east/north (lower) and the west/south (higher) portal resulting in a grade of 5.6 percent down to the east/north.

Two emergency escape adits are included in the design to provide egress from the tunnel in dangerous or life threatening conditions, such as may result from an accident or fire. The adits are at the tunnel third-points, and exit onto the shoulder of the existing road. They are horseshoe-shaped with finished internal dimensions of 10 ft high by 8 ft wide (3 m by 2.4 m).

Tunnel Lining Design Overview

Tunnel stability and lining is achieved with three systems: 1) temporary initial support, 2) final lining, and 3) water management. Although presented as separate systems, the design, construction, and operation of the three are closely related. The structural capacities of the initial support and final lining are independent; with the initial support necessary only during construction, after which the final lining will be utilized. Although it is possible to combine these systems, for this project the independent approach had the advantages of lower overall cost, faster tunnel excavation, and it met project funding goals for the first phase of the work. Construction methods that are rapid, easily mobilized, and flexible are beneficial. In this context, initial support and favors shotcrete and simple rock bolts rather than other materials and methods that are more difficult to mobilize or slower to construct. This paper presents a condensed overview of tunnel lining design; for additional details reference Pease and McKenna (2002a), and Pease (2002b).

Tunnel Initial Support

The function of the initial support is to provide temporary stability and safety for the tunnel opening during construction and prior to installation of the final lining. For overall economics and efficient excavation it was decided that the initial support system would be installed quickly and assumed to be sacrificial, and therefore was not designed for long-term performance. Initial support consists entirely of rock bolts and shotcrete. Although the rock bolts could have been incorporated into the final lining, this would have required more expensive bolts and more difficult installation to account for corrosion protection, and quality assurance and control (QA/QC). With a sacrificial initial support system, the bolts could be installed quickly, without long-term corrosion protection, and a relatively low level of QA/QC could be employed during construction. This system was determined to be less expensive than a more robust initial support system incorporated into the final lining.

Three levels of support were specified as presented in Table 3. Each support level was tied to specific ground conditions and ground classifications (Common Rock for levels A and B, and Poor Rock for level C) in the Geotechnical Design Summary Report. The shotcrete was reinforced with steel fibers, although welded wire fabric was an option for the contractor.

Table 3. Tunnel Initial Support

Support Level	Bolts	Shotcrete	Other
A	10 ft #8 @ 6 ft (3m x 25mm @ 1.8 m) both ways	3 inches (76 mm)	NA
B	12 ft #8 @ 4 ft (3.7 m x 25 mm @ 1.2 m) both ways	4 inches (102 mm)	NA
C	Spot as necessary	4 inches (102 mm)	W10 x 65 Steel Ribs (254 mm x 97 kg/m) Wide Flange @ 4 ft (1.2 m) center to center spacing

Initial support level A was used for the entire tunnel except for the first 30 to 40 feet (9 to 12 m) at each portal. Additionally, spot bolts were added as necessary to address potentially loose blocks and slabs that could fall out. Although steel ribs were pre ordered and available for bad ground (classified as Poor Rock), they were not used.

Tunnel Final Lining

The function of the final lining is to provide long term stability of the opening and to support the water management system. A major consideration in the design is that the completed tunnel be low maintenance. The final lining is structurally independent of the initial support although it is important that it is in good contact with the initial support shotcrete. Cast-in-place (CIP) concrete is required for approximately 50 ft (15 m) at both portals to provide a high level of support due to the lack of confinement, the likely more open character of the joints, and to provide a transition from the portal canopies. For the remainder of the tunnel the contractor was given the option of using either CIP concrete or shotcrete for the final lining. The final lining for the adits is designed to be shotcrete with a spray-on membrane.

The CIP lining was designed to be used in combination with either a PVC sheet membrane or a spray-on membrane at the contractor's option. The contractor has tentatively chosen to use a PVC sheet membrane although he is considering use of a spray-on membrane. The concrete is designed to be 12-in. (305 mm) thick, with one layer of #6 (19 mm dia.) reinforcing bars at 12 in.(305 mm) spacing both ways positioned two in. (51 mm) from the inside surface. This reinforcement position gives the best resistance to bending caused by inward movement of the rock.

The shotcrete lining was designed to be used in combination with a spray-on membrane. This combination was designed to facilitate good lining-to-ground contact and for ease of construction. The shotcrete lining was designed to be nine inches thick (229 mm) and reinforced with either steel fibers, welded wire fabric, or reinforcing bars. Rock bolts are not part of the final lining. This system is unusual in the use of shotcrete by itself without rock bolts, the use of steel fibers for reinforcing, and the use of a spray-on membrane. A key factor in allowing shotcrete was the good quality of the rock mass, which requires only a low level of support, provided it remains intact and does not loosen over time. The shotcrete provides this long-term protection and restraint. Although the shotcrete could be applied in one layer, it was often applied in two to three layers to facilitate adherence to the rock and to fill in areas of overbreak.

Ground Water Control

Although seepage quantity and pressure on the lining are expected to be small, any water penetrating the tunnel lining could result in ice on the inside the tunnel and on the road surface, and could also damage the lining. A three-part water control system has been designed consisting of: 1) drainage system; 2) a water barrier membrane; and 3) formation drains to collect and discharge the water.

Drainage is provided in two locations; first between the initial support and the rock surface and, second immediately behind the membrane. This drainage is provided primarily by one ft wide strip drains, but can include a continuous drainage geotextile for some final lining alternatives as presented in Pease (2002b). Formation drains at the bottom corners of the tunnel will intercept the seepage and transmit it out of the tunnel. They are comprised of a perforated pipe embedded in porous (popcorn) concrete below the outside edge of the sidewalks.

The final line of defense against seepage will be a membrane sandwiched between the initial support and the final lining. The contractor has tentatively chosen to use a PVC sheet membrane, although spray-on membranes are being evaluated.

Emergency and Life Safety Provisions

Design for public safety in potentially dangerous or life threatening emergency conditions, such as from an accident or fire, was a prime consideration in tunnel design. For the Upper Narrows Tunnel, air quality during normal operation is provided with a light ventilation system of jet fans. In emergency situations, safety is provided with two escape adits exiting onto the existing road. The escape adits are unusual, but fit with tunnel geometry and available power limitations. (Gonzalez and Pease, 2001). Additional life safety provisions include emergency pull boxes, video monitoring, and variable message signs. Monitoring will be at the control facility for the Hanging Lakes Tunnels near Glenwood Springs. This arrangement avoids an independent expensive onsite monitoring facility, and utilizes tunnel safety expertise at the existing Hanging Lakes facility.

Tunnel Construction

Tunnel excavation was with drill-and-blast methods using multiple drifts. There were three drifts in the top heading and three in the bottom bench, with initial support installed in each drift as excavation proceeded. The central drift of the upper heading was excavated first, effectively serving as a pilot tunnel with the two side slashes of the top heading following. Bench excavation was led by the center bottom heading to followed by the small side slashes to facilitate maintaining rock mass quality at the excavation line. Additionally, precision blasting methods were used at the final cut lines to result in a low level of damage to the remaining rock.

The tunnel was excavated and initial support installed without any significant complications. In general the rock mass behaved as expected. There was occasional overbreak to horizontal and vertical joints, but in general this was limited to less than two ft (0.6 m). There were only two instances where the overbreak was significantly more than two feet, with the largest approximately 4 ft (1.2 m) behind the excavation line and 20 ft wide by 30 ft long (6 m x 9 m). These large overbreaks were associated with elongated rock wedges defined by horizontal and vertical joints.

Level A initial support was used throughout the tunnel except for 30 ft and 40 ft (9 m and 12 m) at the West and East portals, respectively, as designed. Additionally, spot bolts were used to stabilize potentially loose blocks. Special attention was given to horizontal joints slightly above the tunnel crown that created thin slabs, and wedges formed by combinations of horizontal and vertical joints.

Installation of the drainage wicks and shotcrete over the drainage wicks was challenging. The first difficulty was that it was hard to attach the wicks to the rock prior to the application of shotcrete. Small expansion bolts with large washers were used. A second problem was that shotcrete did not adhere well to the outside of the strips. This was solved by attaching chicken wire to the outside of the strips as a substrate to temporarily hold the wet shotcrete. A third complication was that the excavated surface was highly irregular and when the strips conformed to the surface it created localized reverse gradients, especially in the arched tunnel crown. To address these situations a change was made to allow an initial layer of shotcrete to be placed against the rock surface to smooth out the attachment surface. There is however, still two to three inches (51 to 76 mm) of shotcrete over the strip drains. This change not only made installation of the strip drains easier because pneumatic nails could be used for attachment, but also reduced the frequency of localized reverse gradients in the strip drains.

GROUND NAIL WALLS

Ground nail walls (GNWs) were used to limit the extent of cuts in soil (talus) on the up hill side of the road. To improve the visual appearance of the walls, the shotcrete face was colored, sculpted, and stained. These visual treatments were done at the direction of a landscape architect from the Forest Service. Unlike most other types of retaining walls, GNWs can be constructed incrementally from the top down without temporary ground support, significantly reducing cut volumes as well as the expense of a temporary shoring system. Additionally, GNWs can be constructed in more confined areas because the need for a stabilizing footing is eliminated.

Construction of the GNWs in talus can challenging for drilling and grouting of the nails because of the combination of hard blocks, noncohesive sand, and potentially voids. These problems are not unique to the Upper Narrows Project and have been encountered on other projects with similar conditions throughout Colorado. To maintain hole stability, a casing was used to stabilize the drill holes. Additionally, the design made provisions for socks to be used around the nails to contain the grout. However, excessive grout loss was not a problem and the socks were not necessary.

The visual appearance of the GNWs was of great interest to the design team, CDOT, and the United States Forest Service. GNWs are normally faced with shotcrete, which is commonly considered to have an unattractive, artificial look. Other recent projects in Colorado have utilized mechanical stabilized earth (MSE) walls with a block facing in front of the GNW to provide a more appealing finish. This adds substantially to the cost of the wall, however, and requires the GNW to be positioned farther into the hill than otherwise would be necessary. Additionally, the block facing of MSE walls has an artificial appearance that is not desirable for a natural setting, although it is generally considered to be more attractive than shotcrete. For an economical solution, the GNWs in the Upper Narrows Project used sculpted and colored shotcrete. This outer shotcrete facing is applied after construction of the structural portion of the wall is complete. Sculpting consists of mimicking rock joints and features with an artistic trowel finish and potentially staining for accent. Additionally, the shotcrete is colored to match other nearby rock exposures, and stain is applied after application for additional effect. The concept is to mimic the natural rock cliffs in color and character so that the walls blend into the natural surroundings. Figure 4 shows a completed wall between two rock cuts south of the west portal.

Figure 4. Ground Nail Wall and Rock Cuts.

ROCK CUTS

Design

The road cuts are primarily in rock with a few small areas in soil. Portal cuts are up to 160 ft high (49 m) and cuts along the road outside the portal areas up to 100 ft high (30 m). Rock cut design focused on maintaining slope stability, addressing rockfall issues, and visual appearance.

The rock cuts were designed at an inclination of 1 horizontal to 6 vertical. This near vertical face was appropriate because the rock mass was relatively massive with moderately spaced joints with the predominant joint sets sub-vertical. It was expected that the cuts would be heavily influence by the joints and would frequently break back to the joints.

Designs to reduce the incidence and impacts of rockfall include scaling of loose rocks and the use of rockfall mesh, rock dowels, and catchment ditches. After excavation, the face of the cut was scaled to remove loose blocks. Spot dowels were used as necessary to stabilize potentially loose blocks and to reinforce the face. The dowels are 20 to 30 ft long No. 7 (6 to 9 m long x 22 mm) 75 kip/in^2 (517 MPa) epoxy coated with cement grout and centralizers. The contractor chose to use socks around many of the dowels to contain the grout and prevent excessive grout migration into rock joints. Dowel placement was highly irregular and not on any predetermined pattern. On average there is approximately one dowel for every 500 ft^2 (46 m^2) of exposed rock cut face.

The mesh was designed to be applied from the top of cuts to within 30 ft (9 m) of the roadway thereby limiting the effective rockfall cut height to 30 feet (9 m). The catchment ditches are 14 ft (4 m) wide (from the edge of the pavement and shoulder) and are a modified version of the Ritchie ditch (Ritchie, 1963). The catchment area is often wider than the minimum because of the additional setback resulting from the saw-tooth geometry of the cut toe in plan view.

Several measures were used to improve the visual appearance of the rock cuts and present a more natural appearance. Within the lower 20 to 30 ft (6 to 9 m) of the rock cuts, half casts were required to be removed to reduce the artificial look. However, the blasts broke back to existing joints, and removal of half casts was not necessary. To be less obvious, the bottom of the rockfall mesh was terminated in a variable pattern, not horizontal, and the mesh was used only above the predominant view window which is the first 20 feet (6 m) above the highway. Additionally the mesh is dark colored, and it is pinned to the rock conforming to the cut profile and irregularities, thereby avoiding a "spider-web" appearance.

East Portal

At the east portal and north of the tunnel, the two sub-vertical joint sets were very persist and continuous and controlled the final cut geometry. The first joint set is very well developed, smooth, planar, and prevalent, and strikes approximately 20 degrees to the road. A complementary set is poorly developed and less prevalent, and strikes approximately 70 degrees to the road. Both joint sets commonly dip at 80 to 90 degrees toward the road. They would be ideal joints to control the rock cuts if they were parallel with the road, rather than striking at an angle to the road.

As constructed, the cuts follow these two joint sets in a saw-tooth configuration when looked at in plan view. The cut follows a smooth joint surface sub-parallel (20 degrees off) with the road, then cross back on the complementary joints in a jagged fashion until another sub-parallel joint is picked up. Similarly, the cut face for the east portal of the tunnel follows the second joint and is angled 20 degrees off of perpendicular to the road. There are at least three such "teeth" in the cut face. Figure 5 shows the jointing at the East Portal. The shotcrete patch is visible, but the rockfall mesh had not been applied when this photo was taken.

Figure 5. East Portal Rock Cut.

This situation has both advantages and disadvantages. The saw-tooth profile resulted in a rock cut with a more natural appearance than a smooth planar cut breaking to a line of drill holes. The smooth planar joint surfaces are often free of loose blocks resulting in a low probability for rockfall. In some areas the rockfall mesh was eliminated. On the negative side, the joints resulted in significant overbreak, which was 25 to 30 percent of the theoretical minimum cut volume. Additionally, a major joint close to the blast trim line led to a large slab failure immediately after a blast.

There is an anomalous area high in the apex of the east portal cut with unfavorable joint geometry requiring special measures. The joints in this area are more frequent and they dip into the mountain, not out, resulting in potential toppling instabilities. Additionally, frequent sub-horizontal joints create a blocky condition like sugar cubes stacked at an angle. This area followed a joint swarm and topographic inset identified during explorations and design, and dubbed "the cleft". A large number of rock dowels, hand drilled with jacklegs, were used to stabilize this area. Additionally, reinforced shotcrete facing was used to retain the ground between bolts in a small zone of especially low quality rock mass.

West Portal

At the west portal (south of the tunnel) the sub-vertical joint sets are nearly parallel with and perpendicular to the road. As a result of this pattern, the portal cut is parallel with the road without a saw-tooth profile and the portal face is perpendicular to the tunnel. However, the joints are poorly developed and not continuous in comparison with the west portal, and the cut faces are not as smooth and planar.

West Portal Rock Rib

At the west portal there is a thin web of rock separating the tunnel and the pre-existing exterior rock face, as shown on Figure 6A. This web is approximately 24 ft thick (7 m) which equates to approximately half of the excavated tunnel diameter. Complicating this condition there are several prominent outward dipping sub-horizontal joints and a few vertical joints, creating a potential for sliding and toppling failures, and even collapse of the rib, as shown on Figure 6B. These joints are clearly visible in the cut face. During design there was concern for stability of the web especially from joint movement resulting from a combination of factors including the increase in vertical stress, reduction of confinement, and blasting vibrations from tunnel excavation. Although the joints were identified during explorations, their complete pattern and precise orientations could not be determined before exposure following excavation of the portal face cut.

The rock web was stabilized with a approximately 12 post-tensioned rock bolts with locations identified in the field after completion of the portal cut, but prior to tunnel excavation. They were placed at slight downward orientations crossing the key joints at acute angles and with the anchor zone beyond the joint. The bolts are epoxy coated No. 10 (32 mm) bars having a yield strength of 60 kips/in^2 (414 MPa) with approximate 15- to 30-ft free zones (4.5 to 9.1 m) and 10- to 15-ft anchor zones (3.0 to 4.6 m). The anchor zone was secured with epoxy grout and the free zone with neat cement grout. As evidence of joint continuity and condition, during drilling of the bolt holes drill water flowed out of independent open holes drilled from the perpendicular face. Three bore hole extensometers with three anchor points each were also installed across the joints at acute angles and monitored to verify that there was no movement. No movements of the joints were detected during construction.

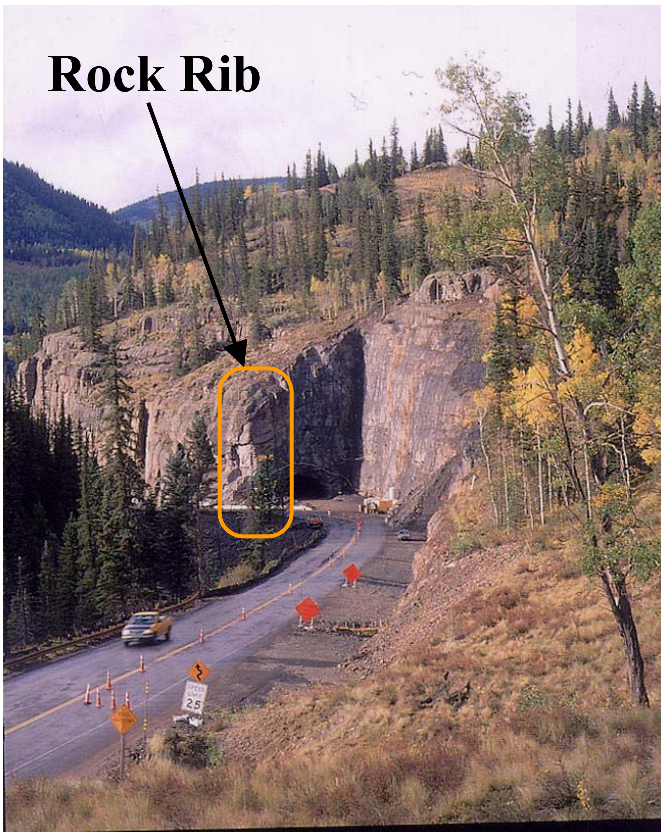

Figure 6A. West Portal Area.

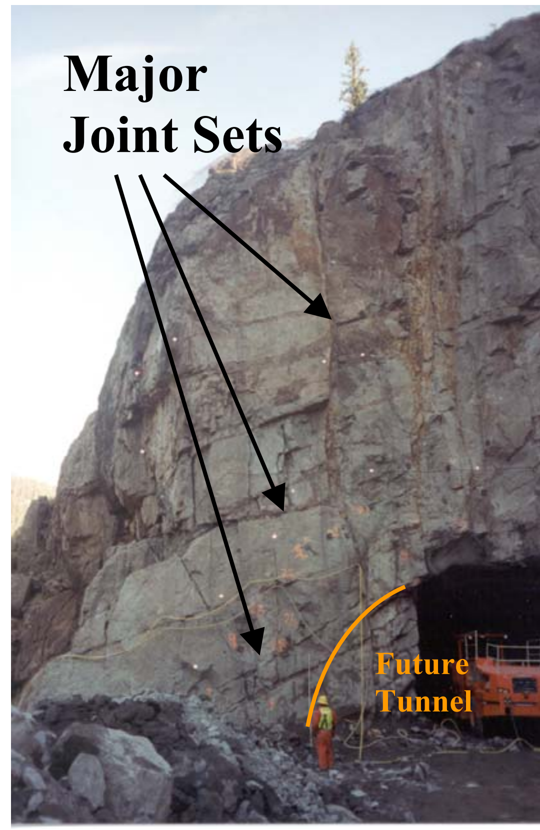

Figure 6B. West Portal Rock Rib.

CONSTRUCTION

Project construction is being conducted in two phases, each with an independent contract and bid. Phase One was constructed by Kiewit Western Co. with work beginning in September 2000 and complete in January 2003. The successful bidder for Phase Two is ASI RCC, Inc. with work beginning in the spring of 2003 and to be complete in the spring of 2004.

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GEOLOGIC CONSIDERATIONS FOR THE DESIGN AND CONSTRUCTION OF THE PLATEAU CREEK TUNNELS, COLORADO

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ABSTRACT

Two 10-ft-diameter (3 m) hard rock tunnels were constructed to replace portions of 14.5 mi (23.3 km) of deteriorated and undersized pipeline as part of the Ute Water Conservancy District's Plateau Creek Pipeline Replacement Project. The project is located about 26 mi (41.8 km) northeast of Grand Junction adjacent to the Colorado River and Interstate 70. The project parallels a narrow, deeply incised canyon that is a tributary to the Colorado River. A feasibility study indicated that two tunnel sections along the pipeline alignment would provide a cost-effective alternative to an open-cut pipeline for traversing the steep canyon along the pipeline replacement route. The tunnels, completed in 2001, include the 3400-ft-long (1036 m) Lower Mesa Tunnel and the 10,000-ft-long (3048 m) Lower Canyon Tunnel.

The tunnels were constructed in interbedded sandstones, siltstones, and shales of the Mesa Verde Group. A Geological/Geotechnical Data and Baseline Report was prepared for the project that identified and quantified the geologic factors affecting the construction of the project. Some of these factors included: potential for encountering methane within the tunnel due to coal beds below the tunnel, potential for groundwater inflows, variable conditions at the face due to the interbedded nature of the bedrock, potential for rockfall hazards at steep portals, the potential for

swelling and slaking of the shale units, and the occurrence of hard inclusions within the sandstone units.

Both tunnels were excavated using a tunnel boring machine. A world record was set during excavation of the tunnels with 219 ft (67 m) of tunnel excavated and supported in one 10-hr shift.

PROJECT DESCRIPTION

General

The Ute Water Conservancy District (Ute Water) identified a need to replace their existing water supply pipeline known as the Plateau Creek Pipeline in order to meet growing water demands in the Grand Valley. The replacement pipeline alignment is nearly identical to the previous alignment. The alignment is approximately 14.5 mi (23.3 km) long and transmits water from Jerry Creek Reservoirs to the Ute Water water treatment plant. The majority of the pipeline alignment follows Plateau Creek from the Jerry Creek Reservoirs to near the Plateau Creek confluence with the Colorado River. Approximately 2000 ft (610 m) upstream of the confluence, the pipeline turns and penetrates a high mesa separating the Plateau Creek and Colorado River valleys via a tunnel. From the Colorado River side of the tunnel portal, the existing pipeline alignment continues in a southwesterly direction approximately parallel to Interstate Highway 70 to the Ute Water water treatment plant. The two new tunnel sections along the alignment include the approximately 3400-ft-long (1036 m), 10-ft-diameter (3 m) Lower Mesa Tunnel that replaces the pipeline alignment through the smaller and shorter Mesa Tunnel, and the new approximately 10,000-ft-long (3048 m), 10-ft-diameter (3 m) Lower Canyon Tunnel. The Lower Canyon Tunnel replaces approximately 15,200 ft (4633 m) of pipeline alignment through the steepest and most sinuous part of the pipeline alignment. A project vicinity map and project location map are shown on Figure 1.

The previous pipeline consisted of 24- to 42-in-diameter (61 to 107 cm) prestressed concrete. The new water supply pipeline is 48- to 54-in-diameter (122 to 137 cm) steel.

Site Description

The project area is marked by deeply incised canyons exposing flat to gently dipping beds of sandstone, siltstone, and shale and high basalt capped mesas. The canyons of Plateau Creek and the Colorado River in the project area expose approximately 1200 ft (366 m) of interbedded sandstone, siltstone, and shale between the water level and the top of the mesas, as shown on Figure 2. The steep canyon walls consist of near vertical sandstone and siltstone cliffs separated by moderately sloping (25 to 30 degrees) less resistant shale layers. Surficial deposits of alluvium and colluvium mantle the lower portions of the canyon walls. Landslide deposits and debris fans are located on the steep canyon slopes within the project area, but are not encountered in the immediate vicinity of the tunnel alignments.

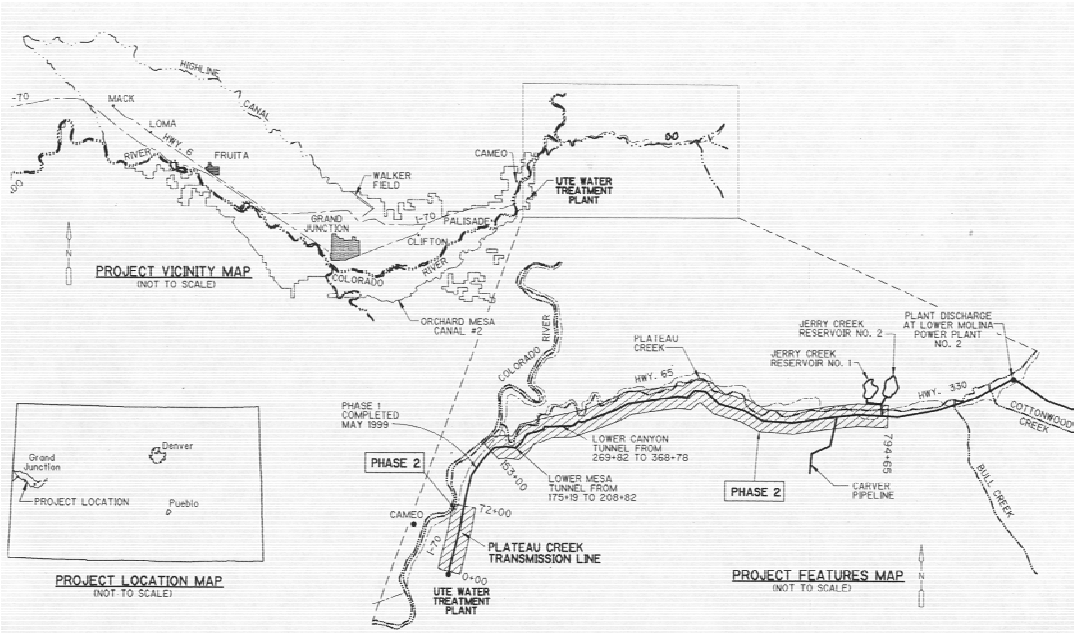

Figure 1. Project Location Map

Figure 2. View of Plateau Creek Canyon, looking downstream

The vegetation in the project area is generally sparse, with some grasses and shrubs in the valley bottoms and scattered sage, scrub oak, small pines and junipers covering the lower slopes.

Project Configuration

The new Lower Mesa Tunnel is located between the Plateau Creek Canyon and Colorado River (DeBeque) Canyon. The tunnel penetrates an arm of the mesa separating the two canyons. The west and east portals of the Lower

Mesa Tunnel is located approximately 300 ft (91 m) above creek level at elevations 5050 and 4932 ft respectively. The tunnel grade drops approximately 118 ft (36 m) from west to east, which is approximately 3.4 percent in the east (upstream) direction. The height of overburden materials over the tunnel roof ranges up to approximately 650 ft (198 m). The 10-ft-diameter (3 m) tunnel cross section includes a 48-in-diameter (122 cm) steel pipe offset to one side of the tunnel to provide access for maintenance and inspection of the pipeline.

The Lower Canyon tunnel is located along the south canyon wall of Plateau Creek. The west and east portals of the tunnel are located approximately 10 to 20 ft (3 to 6 m) above the creek level at elevations 4854 and 4969 ft, respectively. The tunnel grade drops approximately 115 ft (35 m) from east to west, which is approximately 1.2 percent in the west (downstream) direction. A vertical and horizontal curve in the alignment was necessary to provide adequate cover over the tunnel roof at the location of an incised draw near the mid-point of the alignment. The height of overburden materials over the tunnel roof ranges from approximately 80 ft (24 m) at the incised draw to over 700 ft (213 m). The 10-ft-diameter (3 m) tunnel cross section is similar to that of the Lower Mesa Tunnel.

BACKGROUND INFORMATION

The existing Mesa Tunnel currently contains the Plateau Creek pipeline and was constructed in the mid 1960's by conventional drilling and blasting methods. Except for a short distance from the portals, the tunnel is unsupported. The existing Mesa Tunnel was excavated to a 5-ft by 7-ft 1.5 to 2.1 m) cross section and a 24-in-diameter (61 cm) steel pipe placed on the floor of the tunnel (GEI, 1995, GEI, 1996). The pipe was backfilled to some depth over the top of the pipe with bedding material. The west tunnel portal is at elevation 5145 ft and the east portal at 5220 ft with a length of 2560 ft (780 m) resulting in a drop of 75 ft (23 m) (2.9 percent) from east to west. The west portals of the proposed and existing tunnels are approximately 250 ft (76 m) apart but the alignments diverge by 17 degrees separating the east portals by 1200 ft (366 m).

REGIONAL SETTING

The project is located on the southwestern edge of the Piceance Basin northeast of the Uncompahgre Uplift. The predominant bedrock lithologies in the project area are interbedded sandstones, siltstone and shales with some carbonaceous shale and coals of the Cretaceous-aged Mesaverde Group (Donnell, Yeend, and Smith, 1984). Bedrock of the Wasatch Formation is exposed in the upper reaches of the Plateau Creek Canon above the DeBeque cutoff (intersection of Colorado Highway 65 and County Road 45.5 Road) and will not be encountered along the proposed tunnel alignments. The bedrock units in the project area dip gently downward to the northeast into the Piceance Basin at less than 5 degrees.

The Mesaverde Group in the project vicinity consists primarily of non-marine deposits laid down during the slow withdrawal of the Mancos sea (Burger, 1959). The Mesaverde intertongues with and grades downward with the Mancos Shale.

The Mesaverde Group consists of at least 12 different lithologies from six major depositional environments (Burger, 1959). There are rapid local variations in thickness of units within the Mesaverde group. The lower sequence of the Mesaverde is predominantly marine sandstones, shale and impure sandy limestone and minor beds of coal, and the upper sequence is mixed marine and continental rocks consisting of conglomerate, sandstone, siltstone, shales, freshwater limestones and coal (Burger, 1959). The portion of the Mesaverde Group exposed in the project area is generally the Farrer and Neslen Facies of the Price River Formation of the Mesaverde Group (Young, 1959).

Coal beds are common in the lower portion of the Group but are not as common in the upper, less carbonaceous portions. Locally the Roadside Mine, located two miles west of the west end of the tunnel alignment, produces coal from the Cameo B Seam of the Bookcliffs Coalfield.

Surficial geological units identified in the project area include alluvial, colluvial, debris fans, landslides deposits, and fill (Whitney, 1981).

The structure of the bedrock bedding in the project area is generally flat. Regional bedrock dip is generally less than five degrees downward to the northeast (Grout and Verbeek, 1985). Three major discontinuities in the bedrock affect rock mass structure in the project area. These include nearly horizontal bedding planes in interbedded sandstone and shale bedrock and two nearly vertical joint sets oriented along strike directions of east-northeast and north-northwest (Grout and Verbeek, 1985). Several high angle normal faults are mapped in the general vicinity of the project. The closest mapped faults are a series of normal faults approximately 10 mi (16 km) north of tunnels (Donnell, Yeend, and Smith, 1984).

SITE GEOLOGY

General

Geologic evaluations and field investigations were performed to identify geologic or physical conditions that could potentially affect the design and/or construction of the proposed tunnels. The evaluations included review of available geologic literature and maps for the project area, evaluation of air photographs for the project area, and site geologic reconnaissance and mapping. Special emphasis was placed on defining the nature of the geologic materials and discontinuities, including: bedrock stratigraphy and structure; the attitudes and engineering characteristics of joints, potential faults or shears; the nature, thickness, and areal extent of surficial deposits; and potential geologic hazards, including evidence of instability of the natural slopes in the portal areas. Site reconnaissance of the portal areas were performed before the subsurface exploration program to assist in the identification of the optimum locations for the tunnel portals.

Subsurface Investigations

Two subsurface exploration programs, consisting of exploration borings, were performed to investigate the subsurface conditions for the two tunnel alignments. The purposes of the borings were to evaluate the subsurface materials along the tunnel alignment and portal locations, identify the general engineering properties of the subsurface materials, and obtain samples for laboratory testing.

Because of the difficult access, heli-portable drill equipment was used to obtain access to the majority of the boring locations. Rock coring was performed using NQ-size coring techniques.

<u>Lower Mesa Tunnel</u> - The subsurface exploration program for the Lower Mesa Tunnel was performed in the fall of 1998. Five borings were drilled at selected locations along the proposed tunnel alignment. Total drill footage was approximately 810 ft (247 m). Selected borings were drilled inclined to obtain additional information about the jointing characteristics of the rock mass.

<u>Lower Canyon Tunnel</u> - The subsurface exploration program for the Lower Canyon Tunnel was performed in the winter of 1999. Seven borings were drilled at selected locations along the proposed tunnel alignment. Total drill footage was approximately 2000 ft (610 m). Selected borings were drilled inclined to obtain additional information about the jointing characteristics of the rock mass. Piezometers were installed in the deepest two borings.

Packer tests were performed in bedrock to estimate the permeabilties of the various bedrock strata. Packer tests were done at approximately ten to 100 ft (30 m) intervals at two different tests pressures.

Testing Program

A total of over 50 representative samples of the sandstone, siltstone, and shale rock formations were selected for laboratory testing based on field observations and descriptions of the rock core. The purpose of the rock core tests was to characterize the bedrock parameters for analysis and design of the excavation and support requirements of the proposed tunnel. The laboratory tests were performed by the Earth Mechanics Institute, Colorado School of Mines, Golden, Colorado and Advanced Terra Testing, Lakewood, Colorado.

Laboratory tests consisted of bulk density, uniaxial compressive strength, point load, Brazilian tensile strength, Young's modulus, Poisson's ratio, punch penetration, Cerchar abrasivity index, swell pressure, slake durability Atterberg limits, petrographic analysis, and joint direct shear.

GROUND CHARACTERIZATION

Bedrock units in the project area include the sandstones, siltstones, and shales of the Mesaverde Group. Because of the variety of depositional environments in the Mesaverde Group, the type and lateral continuity of the rock types can vary appreciably over short distances, both vertically and horizontally, and may even be abrupt. Because of this variability, projection of bedrock units, even over short distances, is difficult.

Weathering of the bedrock units is variable across the project site. In general, the sandstone units are more resistant to weathering and tend to form the massive cliffs in the project area, whereas the finer-grained materials - the siltstones and shales - tend to be less resistant to weathered and from the slopes in the project area. In general, the depth of weathering of the bedrock units is a function of rock type and frequency of jointing within the rock types. In general, the depth of weathering in the sandstone units is less than that for the siltstone and shale units because of the frequency of jointing within these finer-grained units.

Sandstone Units

Based on the geologic mapping and subsurface exploration information, the sandstone units comprise approximately 70 percent of the stratigraphy of the Mesaverde Group. The typical stratigraphy along the tunnel alignment was presented as shown in Figure 3. The sandstones generally range from gray, fine-grained, argillaceous, quartzose sandstones to light brown, fine- to medium-grained, arkose sandstones. Iron staining permeating the rock mass will result in a reddish-brown color. Petrographic analyses performed on selected sandstone samples indicate that the sandstones are composed of 35 to 60 percent quartz; 40 to 45 percent muscovite, plagioclase, calcite, and microcline; and 10 to 25 percent pore space. The high percentage of pore space appears to be related to the dissolving of the calcite cement.

Bedding ranges from laminated (less than 1 in) to massive (tens of ft). Individual sandstone beds range from less than 1-ft (30 cm) to greater than 50 ft (15 m) thick with most beds between 10 and 20 ft (3 to 6 m) thick. The moderately weathered to fresh sandstone is moderately hard to hard. Unconfined compressive strengths of the sandstone samples tested ranged from less than 3000 psi (210 kg/cm^2) to greater than 26,000 psi (1828 kg/cm^2), with an average value close to 10,000 psi (703 kg/cm^2).

Observed discontinuity spacings in the sandstone rock mass generally ranged from closely spaced (less than 0.3 ft) in the laminated and thinly bedded sandstones to widely spaced (greater than 1 ft) in the more massive sandstone beds. RQD's of the sandstone units ranged from 58 to 100 percent, with an average RQD of about 90 percent. Joints observed in cores were generally tight or slightly open with iron staining or very thin calcite fillings. Observed joint surfaces were generally planar and slightly rough to stepped.

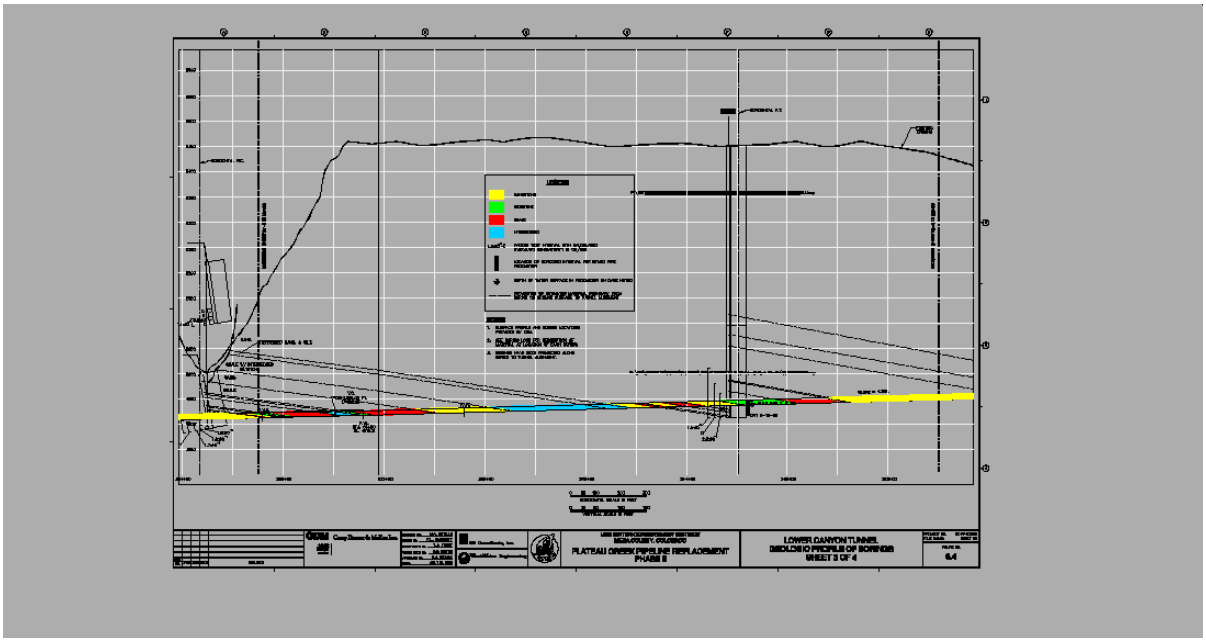

Figure 3. Typical Presentation of Stratigraphic Units Along Tunnel Profile

Iron concretions have been observed in the sandstone units. These features tend to be oriented in the direction of bedding, subrounded to lobate, typically less than 3 in (8 cm) in diameter, with maximum observed dimensions approximately 1 ft by 6 ft (0.3 by 1.8 m).

Siltstone Units

Based on the geologic mapping and subsurface exploration information, the siltstone units comprise approximately 15 percent of the stratigraphy of the Mesaverde Group. The siltstones are generally medium to dark gray with bluish streaks with varying amounts of fine silt, sand and clay-sized particles. Petrographic analyses performed on a selected siltstone sample indicated that the siltstone was composed of approximately 90 percent clay minerals and approximately 10 percent quartz and feldspar.

The siltstones are laminated (less than 1 in), thinly bedded (1 to 6 in) (2.5 to 15 cm) to distorted bedding from bioturbation and/or soft sediment deformation. Individual siltstone beds range from less than 1-ft (0.3 m) to greater than 10 ft (3 m) thick with most beds between 2 and 5 ft (0.6 and 1.5 m) thick. The moderately weathered to fresh siltstone is moderately hard to very hard. Unconfined compressive strengths of the siltstone samples tested ranged from 16,000 psi (1125 kg/cm^2) to greater that 34,000 psi (2390 kg/cm^2), with an average value of about 24,000 psi (1687 kg/cm^2).

Observed discontinuity spacings in the siltstone rock mass were generally very closely spaced (less than 0.1 ft) in the laminated and thinly bedded units. RQD's of the siltstone units ranged from 25 to 100 percent, with an average RQD of about 85 percent. Joints were generally tight with iron staining. Observed joint surfaces were generally smooth and planar to undulating.

Shale Units

Based on the geologic mapping and subsurface exploration information, the shale units comprise approximately 10 percent of the stratigraphy of the Mesaverde Group. The shales are generally gray and dark gray with varying amounts of fine sand, silt, and clay-sized particles. Petrographic analyses performed on a selected shale sample indicated that the material was composed of approximately 97 percent clay minerals and approximately 3 percent. A previously performed x-ray diffraction analysis of two shale samples indicated that the shale is composed of about 45 to 50 percent quartz, 0 to 11 percent calcite and dolomite, 0 to 6 percent feldspar, and approximately 40 percent clay minerals with the clay fraction composed of 70 to 80 percent smectite/illite, approximately 15 percent kaolinite, and approximately 5 to 10 percent chlorite.

The shales are thinly laminated to very thinly bedded. Individual shale beds range from less than 1-ft (0.3 m) to about 8 ft (2.4 m) thick with most beds between 1 and 4 ft (0.3 and 1.2 m) thick. The moderately weathered to fresh shale is moderately soft to very hard. Unconfined compressive strengths of the fresh shale samples tested ranged from 10,000 psi (703 kg/cm^2) to greater that 36,000 psi (2531 kg/cm^2), with an average value of about 22,000 psi (1547 kg/cm^2). Point load tests of the thinly laminated and/or weathered shale samples ranged from 150 to 300 psi (11 to 21 kg/cm^2).

Observed discontinuity spacings in the shale rock mass were generally very closely spaced (less than 0.1 ft) (<3 cm) in the laminated and thinly bedded units. RQD's of the shale units ranged from 20 to 100 percent, with an average RQD of about 70 percent. Joints were generally tight with iron staining. Observed joint surfaces were generally smooth and planar to undulating with some randomly-oriented slickensided surfaces observed.

An interesting finding of the laboratory testing program was that the density and strength (both uniaxial compressive and tensile) of the sandstone samples (on average 134 pcf, 10,100 psi, and 650 psi, respectively) (2146 kg/m^3, 710 kg/cm^2, and 46 kg/cm^2) were less than that for the siltstone (on average 152 pcf, 23,600 psi, and 1980 psi, respectively) (2435 kg/m^3, 1659 kg/cm^2, and 139 kg/cm^2) and shale (153 pcf, 22,000 psi, and 1200 psi, respectively) (2451 kg/m^3, 1547 kg/cm^2, and 84 kg/cm^2) samples tested. The lower density and lower strength of the sandstone is attributed to the well sorted structure of the sandstone and mostly dissolved calcite cement. The higher density and higher strength of the siltstones and shales is attributed to the more closely packing and consolidation of grains within these materials. In regard to the shale samples, only those samples that were competent enough were tested. Shale samples that were not competent could not be tested, therefore sample bias with respect to the shale samples must be considered.

Bedrock Discontinuities

Bedrock discontinuities include those features within the rock mass that form planes, surfaces, or other features that interrupt or separate the otherwise intact rock mass. These include joints and fractures, bedding planes, faults and shears, and other depositional features. A description and general characteristics of the bedrock discontinuities are discussed in the following sections.

Two main joint sets are present in the project area. These joint sets are near vertical and strike east-northeast and north-northwest. These two sets tend to separate the bedrock units into more or less irregular cube-shaped blocks. The joint sets are more closely spaced in the harder, finer grained siltstone and shale materials and are more widely spaced in the softer, massive sandstone units. In general the joint surfaces are smooth to slightly rough and planar to undulating. Many surfaces have calcite or iron staining.

A separate discontinuity set includes randomly-oriented slickensided surfaces within the shale units. These are discussed below.

Bedding plane joints are those discontinuities that form along bedding breaks or planes. Typically bedding planes form due to mineralogical or structural weakness in the bedding or due to dissimilarities in the rock composition or structure. The bedding in the project area, and hence the bedding plane joints, dips approximately five degrees to the northeast. Bedding plane joints are most pronounced in the laminated shale units due to the presence of laminated clay particles. Bedding plane joints are less pronounced in the siltstone units due to disturbance of the bedding and nearly absent in the sandstone units due to the massiveness of the bedding.

No fault or shear features have been identified in the project area.

Characteristics of the Shales

As was discussed above, slickensides were observed in the shale units encountered in the geologic mapping and subsurface exploration program. In general, the slickensides are confined to the shale beds, although some slickensides have been observed in other bedrock units. The slickensides are randomly oriented with gentle to steep dips and smooth to striated irregular surfaces. The persistence (continuity) of individual slickensides is low (less than 2 to 3 ft). Because of the nature of the slickensides and surrounding rock, it was not possible to obtain adequate quality samples to test the shear strength and deformation properties of the slickensides.

Because of the low shear strength of the slickensided surfaces, it is anticipated that the presence of the slickensides in the shale units will affect the stability and ground support requirements of the tunnel excavations and cut slopes. Where slickensided surfaces daylight into the excavations, rock blocks or wedges could form which will likely require stabilization to prevent the blocks from sliding into the tunnel excavation. The locations and sizes of potential rock blocks will be variable because of the randomness in the orientation and dip of the slickensided surfaces.

Combustible or Toxic Gases

Although the existing Mesa Tunnel did not record any explosive or toxic gasses during construction in the mid 1960's, current and historic coal mines in the area have reported significant methane emissions. The Mine Safety and Health Administration (MSHA), Colorado Coal Division reportedly makes quarterly inspections of all underground coal mines operating in the Plateau Creek area of the Piceance Basin, and has noted that methane emissions from the bedrock are a regular problem. The Roadside North Mine, discussed briefly above, presently records methane emissions of about 0.5 to 0.7 percent with the ventilation system supplying about 120,000 ft^3/min of "fresh" air.

Studies by the United States Geological Survey (Choate et. al., 1981) have indicated "the Piceance basin, particularly the area southeast of a line connecting Rio Blanco and Mack, is the gassiest coal region in the western United States." Another USGS study (Eager, 1978) reported on the findings in a deep borehole located very near to the east portal of the proposed Lower Mesa Tunnel. Gas emissions were noted in the coal seams, located considerably below the proposed tunnel construction, but it was reported that for pressure gradients lower than 0.433 psi/ft, that the gases had a tendency to flow upward, with the gradient in the Plateau Creek area reported to be about 0.37 psi/ft, indicating a potential for upward migration in the project area.

Based upon this project area evidence, the Plateau Creek tunnels were classified as potentially gassy according to the criteria in the Code of Federal Regulations, Title 29 - Labor, Part 1926 - Subpart S - Underground Construction, Caissons, Cofferdams and Compressed Air, Paragraph 1926.800.h.1.ii - Potentially Gassy Operations.

Hard Inclusions/Concretions

As was discussed above, hard to very hard, subrounded to lobate concretions have been observed in sandstone bedrock units. This is particularly true at west portal of lower canyon tunnel. In general, the concretions tend to be oriented in the direction of bedding, subrounded to lobate, typically less than 3 in (8 cm) in diameter, with maximum observed dimensions approximately 1 ft by 6 ft (0.3 to 2 m).

DESIGN CONSIDERATIONS

Rock Mass Classifications

Two rock mass classification systems, the rock mass rating (RMR) system or Geomechanics Classification (Bieniawski, 1989) and the Q-system, were used to evaluate the anticipated ground conditions and to estimate support requirements along the proposed tunnel alignments. In addition, the RMR system results were used to estimate in-situ rock mass shear strengths and modulus of deformation for the various materials using available empirical relationships (Hoek and Brown, 1997).

Input to the rock mass classification systems included information gathered from geologic mapping, subsurface exploration program, and laboratory testing program.

Table 1 summarizes the average values of RMR and Q under low and high overburden conditions for each of the three general rock types. It should be noted that these are average values. Lower values will control the rock support needed to support the tunnel opening.

Table 1. Summary of Average RMR and Q Values by Rock Type

Rock Type	RMR	Description	Q	Description
Sandstone	64-76	Good	6.7-57.2	Fair-Very Good
Siltstone	55-72	Fair-Good	2.4-33.3	Poor-Good
Shale	52-62	Fair-Good	1.0-16.7	Poor-Good

Design Analyses

Rock Mass Classification Ground Support Implications - For the rock types and qualities anticipated along the length of the Plateau Creek tunnels, various empirical methods were used to estimate ground support requirements. These methods included the RMR System and the Q-System as mentioned above, the Terzaghi method, the Rock Structure Rating (RSR), the Rock Quality Designation (RQD) Method, and the Heuer Shotcrete Classification Method (Heuer, 1973) (ASTM,1988). In addition, the Bischoff and Smart Method of comparing an internal support system (reinforced rock arch) to an equivalent external structural steel support system was used (Bischoff and Smart, 1975).

For these evaluations, three different rock types were considered, sandstone, siltstone, and shale. For each of these rock types, three different rock qualities were considered - high, low, and average - based upon the rock mass classification results as well as the results of the laboratory tests for strength and deformation characteristics. Two different overburden depth conditions were considered - low ground cover conditions, up to 130 ft (40 m) and high ground cover conditions, up to 640 ft (195 m). Two different excavation shapes were considered, a circular opening to represent excavation by Tunnel Boring Machine (TBM), and a horse shoe-shaped opening representing excavation by either drill and blast or road header.

As input into these empirical methods to estimate ground support requirements, in-situ rock mass properties were estimated by extrapolating the laboratory data on intact rock properties using the methodology developed by Hoek and Brown (Hoek and Brown, 1997). The influence of the ratio of horizontal to vertical stress (K_o) was evaluated. This required the assessment of the most probable value of K_o. For this evaluation, elastic behavior was considered as it applies to the development of lithostatic gravitational stresses in sedimentary deposits, as a function of Poisson's ratio. In addition, the potential influence upon the K_o value, of the eroded valley adjacent to the construction area was considered (Worotnicki, 1969).

<u>Numerical Models</u> - In addition to the empirical evaluations of ground behavior noted above, a numerical model was developed using the FLAC finite difference method of analysis, and a model was developed which modeled the "key blocks" defined by the measured system of naturally occurring discontinuities surrounding the tunnel excavation. These are discussed individually below.

For the FLAC analysis, the various types and qualities of rock were modeled as well as variations in the depth of overburden and in the K_o value. In addition, models were developed for various mixed face conditions with siltstone/sandstone in the crown and shale in the invert, siltstone/sandstone in the invert and shale in the crown, siltstone/sandstone in the majority of the face with a thin layer (~2 ft or so) (0.6 m) of shale located at the springline, and for the condition with an "average" shale in the majority of the face, but with a layer of poor quality shale in the crown, invert or springline.

A total of 73 separate cases were evaluated. As expected, the deeper overburden conditions in combination with the weaker rock materials, especially the layers of poor quality shale, produced the worst ground behavior, with overstressing of the weak material producing plastic deformation of the material into the tunnel opening if left unsupported or unrestrained. This type of stress-controlled instability required the development of a full perimeter structural steel and shotcrete internal support system.

For the non stress controlled mode of ground instability, the instability was controlled by the system of naturally occurring discontinuities in the rock mass forming blocks or slabs of potentially "loose" rock in the crown, bounded by the discontinuities and the perimeter of the excavation. This method of instability was modeled by the "key block" computer model, which defines the size and orientation of the most likely loose block (Goodman and Shi, 1985).

With the three major joint systems measured at the construction site, combined with the bedding plane discontinuities, the most likely "key block" was a slab in the tunnel crown, or a block in the upper haunch of the excavation, with a bedding plane discontinuity as the upper bound, and near vertical joints as the side boundaries. Where the strike of the near vertical joints is near parallel to the alignment of the tunnel, the likelihood of development of this type of loose block is increased. As the tunnel alignment diverges from the strike of the main joint sets, this likelihood decreases. In thinly bedded rock material such as the shales, the development of these loose blocks or slabs is more likely than in the more massive sandstones. Also, the siltstones, being more intensely jointed than the sandstones would be more susceptible to this type of instability.

Ground Support Requirements

Based upon the results of the various methods of analyzing the ground behavior, including both the empirical methods and the numerical models, several types of ground support systems were developed. The support systems are described individually below and illustrated in Figure 4.

Type A: 2-bolt pattern, 4 ft (1.2 m) on center along the tunnel alignment. Typically the host material is sandstone, siltstone, or interbedded sandstone and siltstone with occasional very thin (3/8- to 1-in-thick) (1 to 2.5 cm) shale interbeds in the crown. This is the minimum tunnel ground support to actively support potentially unstable blocks or wedges and to support potentially loose slabs in the crown. (Q>10, RMR>65)

Type B: 4-bolt pattern, 4 ft (1.2 m) on center along the tunnel alignment. Typically the host material is sandstone or siltstone interbedded with 1-in- to 3-ft-thick (0.3 to 1 m) shale beds above the tunnel springline. Bedding planes and/or fracture spacing in the siltstones or shales ranging from 8 to 12 in (20 to 30 cm) create weak planes at or slightly above the tunnel crown. (1<Q<10, 45<RMR<65)

Type C: 4-bolt pattern, 4 ft (1.2 m) on center along the tunnel alignment. Typically the host material is shale or shale interbedded with sandstone and siltstone. The shale exhibits moderately spaced fracturing (4 to 12 in) (10 to 30 cm) and the fracture spacing and/or bedding plane discontinuities indicate a likelihood for displacements along existing discontinuities, resulting in the development of many loose blocks if left unsupported. (0.4<Q<1, 35<RMR<45)

Type D: W4 x 13 steel sets at 4 ft (1.2 m) on center along the tunnel alignment. For use in all rock types as necessary to maintain long term integrity of the tunnel opening in intensely fractured (less than 4 in) (<10 cm) ground and shale with moderately to closely spaced (1 to 12 in) (0.3 to 30 cm) slickensides present from the tunnel springline to at or above the tunnel crown. (Q<0.4, RMR<35).

Support Selection

Table 2 summarizes the designed vs. as constructed support provisions.

The amount of Type D support was reduced by installing a modified Type C support that consisted of “mine straps” in the crown immediately behind the TBM grippers. The mine straps were 6-in-wide (15 cm) steel channel sections, 10 ft (3 m) long that were supported with four rock bolts. The straps were used with and without chain link depending on the condition of the ground. A total of 128 straps were installed in the Lower Mesa Tunnel and 216 straps were installed in the Lower Canyon Tunnel.

As a supplement to the systematic ground support systems, the contract documents and technical specifications included provisions for supplemental ground support in the form of individual

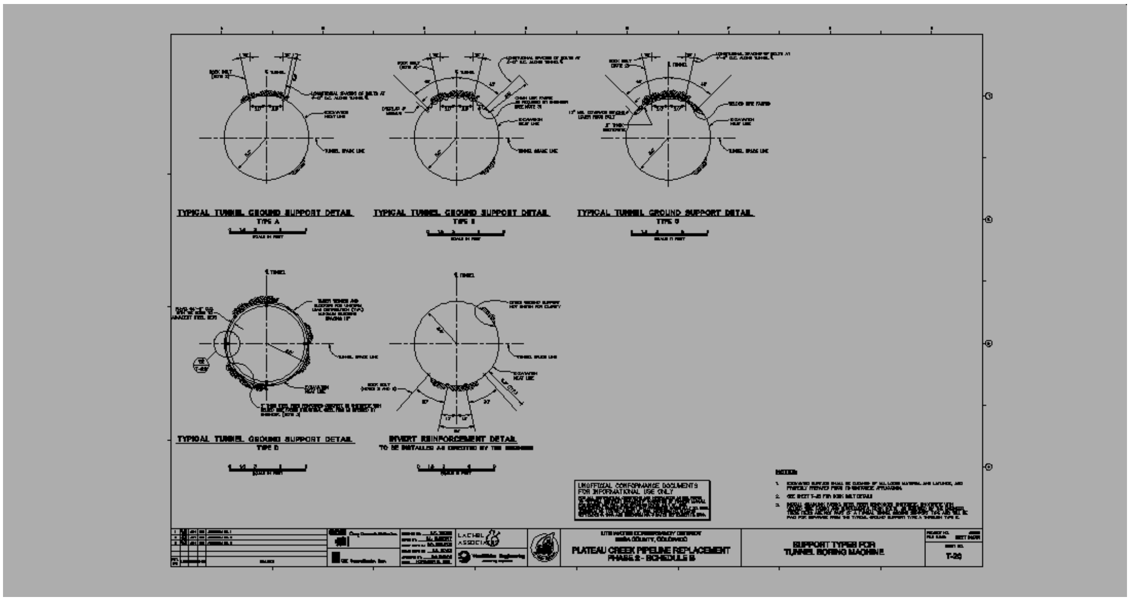

Figure 4. TBM Support Systems

Table 2. Designed vs. As-Constructed Support

Support Type	Baseline Condition, ft (m)	Actual, ft (m)
Lower Mesa Tunnel		
A	1970 (600)	3015 (919)
B	990 (302)	0
C	375 (114)	303 (92)
D	65 (20)	20 (6)
Lower Canyon Tunnel		
A	5700 (1737)	7862 (2396)
B	2400 (732)	51 (16)
C	1600 (488)	1992 (607)
D	300 (91)	12 (4)

rock bolts, chain link fabric, shotcrete, reinforced shotcrete, rock sealant, and invert reinforcement in the form of rock bolts to provide stability against long-term heave of the invert.

Discretionary dry mix shotcrete was installed over 10,050 square ft (934 m^2) in the Lower Mesa Tunnel. The discretionary shotcrete was at least 2 in (5 cm) thick and was applied generally below the spring line in areas of shale material to reduce slaking. Pneumatically applied sealant was applied over 6755 square ft (628 m^2) of generally shale material. Spot rock bolts were installed at 146 locations and chain link was installed over 532 square ft (49 m^2) at the east portal to contain reveling material.

Discretionary dry mix shotcrete was installed over 16,249 square ft (1510 m^2) in the Lower Canyon Tunnel. The discretionary shotcrete was applied in a similar manner to that applied in the Lower Mesa Tunnel. Pneumatically applied sealant was applied over 605 square ft (56 m^2) of generally shale material. Spot rock bolts were installed at 50 locations and chain link was installed over 400 square ft (37 m^2) at the east portal to contain raveling material.

Instrumentation and Monitoring

Instrumentation of ground behavior in either the portal excavations or in the tunnel excavation is not anticipated. However, if unusual ground conditions are encountered unexpectedly in the tunnel, in which the "design" support systems are determined to be inadequate and must be increased in either size and/or frequency of installation, then a simplified method of instrumentation may be implemented to monitor the effectiveness of the "improved" support system.

Other Considerations

Other design and construction considerations included: timing of support relative to the time of excavation, influence of shale interbeds, influence of contact zones where the geologic sequence transitions from one rock type to another, influence of hard concretions, ground water inflow, trafficability of the shale interbeds in the invert of the tunnel, and portal excavation and support considerations.

CONSTRUCTION

Bidding

A total of seven bids were received for the project. The successful low bidder was Barnard/Affholder, joint venture. Affholder was responsible for the underground and portal work and Barnard Construction was generally responsible for the open cut pipeline portions of the project.

Site Preparation/Portal Construction

Construction started in November 1999 and was substantially complete in March 2001 for the tunnel and September 2002 for the entire project. Site preparatory work included providing access to the portal locations and excavation of each of the four portals. Access and portal construction were more difficult at the Lower Mesa Tunnel portals as compared to the Lower Canyon Tunnel portals. Significant rock excavation was required at the east portal of the Lower Mesa Tunnel in order to provide an adequate working area for the tunnel construction. In addition, a Geobrugg rockfall fence was installed above this portal in order to provide protection of the area from rockfalls, as shown in Figure 5.

Figure 5. Construction of the East Portal of the Lower Mesa Tunnel. Note rock excavation partially complete for portal and rockfall fence above rock cut

Tunnels

After excavation and support of the portals, starter tunnels were constructed at the east portal of the Lower Mesa Tunnel and west portal of the Lower Canyon Tunnel where the TBM was launched. The starter tunnels were constructed by conventional drill and blast methods. Each tunnel was constructed to a 14-ft (4.2 m) horseshoe shape for a distance of approximately 40 ft (12 m) (Figure 6). After the initial 8-ft (2.4 m) round, the heading was advance using a jackleg drill to drill each round of 6-ft (1.8 m) blast holes. Approximately 44 holes were drilled for each round with perimeter holes (not loaded) drilled on approximately 8-in (20 cm) centers for smooth wall blasting. The holes were loaded with Magnum, paper-packaged, detonator sensitive emulsion explosives, and stemmed with 12-in (30 cm) clay dummies. Each hole was loaded with 4.15 lbs (9.13 kg) of explosives. Each round was initiated using an Exel LP long delay, non-electric detonation system and Cordtex 18 detonating cord. Figure 8 shows the configuration of the starter tunnel for the Lower Mesa Tunnel in the foreground and the circular TBM tunnel in the background.

After the starter tunnels were excavated and supported, the TBM was mobilized to the tunnel heading to excavate a 10-ft (3 m) diameter tunnel for the remainder of the alignment (Figure 7). The TBM was a 10-ft (3 m) diameter, main beam type Robbins Model #91-155. Prior to arriving on site, the machine was refurbished and upgraded. The cutterhead was rebuilt with 17-in (43 cm) diameter, wedge lock disc cutters. The four original 125 horsepower electric motors were upgraded to 200 horsepower, water-cooled, 480 volt electric motors. The TBM weighed 75 tons and was 155 ft (47 m) long, including trailing gear.

Figure 6. Drill and Blast Starter Tunnel in Foreground and TBM Tunnel in Background

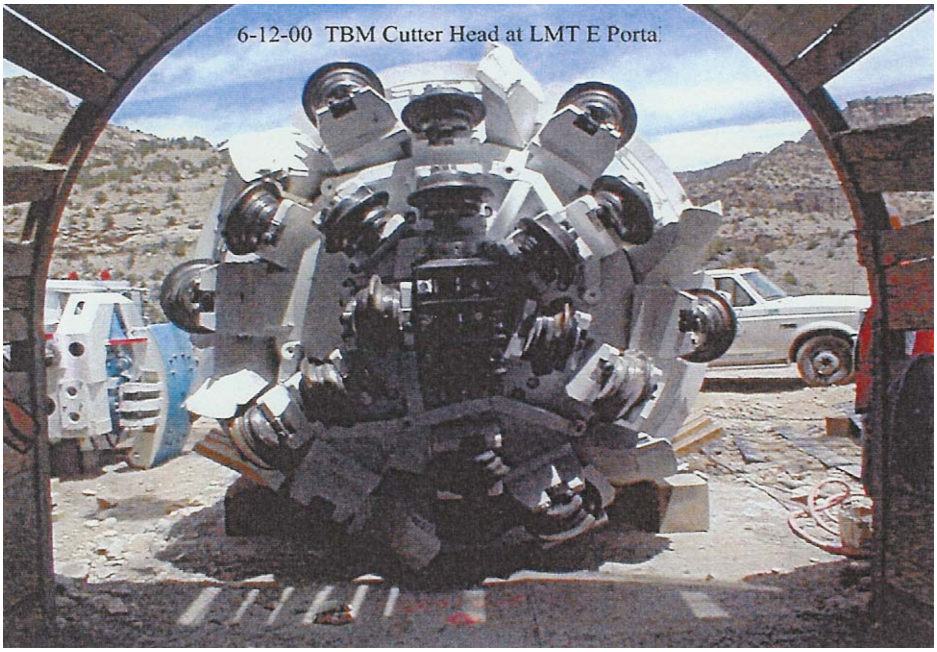

Figure 7. TBM Cutter Head Prior to Initial Launching.

The TBM cutterhead contained 22 cutting discs and 6 muck buckets. The recommended cutter load, per cutter, was 26 tons maximum and 22 tons operating. The cutterhead speed was 11.4 RPM at 800 horsepower. The thrust capacity was 597 tons at 4,500 psi (315 kg/cm^2). After each 46-in (117cm) stroke, the TBM was reset by retracting the grippers, moving the gripper carriage forward, and regripping. The machine was then positioned for the next push.

Muck buckets on the cutterhead picked up the spoil from the bottom of the tunnel and deposited the material on the 18-in (46 cm) wide machine conveyor. The conveyor transported the muck to the trailing conveyor and then to muck cars at the end of the TBM. The spoil was transported out of the tunnel using three trains riding on 24-gauge, 60-lb (132 kg) rail. A moveable California switch was used to allow the trains to pass one another in the tunnel. Three diesel locomotives were used to pull muck trains. The muck cars were 4 cubic yd, lift-off type cars. The muck cars were unloaded at the portals using a crane (Figure 8).

Construction of each tunnel proceeded with very little difficulty. The ground conditions encountered in each tunnel were actually better than expected. Ground conditions were so good in one stretch of the Lower Canyon Tunnel that a world record of 219 ft (67 m) in one 10-hr shift was set for excavating a hard-rock tunnel using a TBM. Groundwater inflows into the tunnel during construction were very minimal to non-existent. No gaseous pockets were encountered in either tunnel.

Figure 8. Muck Disposal at East Portal of Lower Mesa Tunnel.

Tunnel Geology – The tunnel geology was mapped using the full periphery method. The method creates a developed plan by unrolling the circumference of the tunnel to form a plan of the entire exposed surface. In the developed plan view, the tunnel is separated into quadrants that are delineated by the crown, left and right spring lines, and the invert. For a 10-ft (3m) diameter tunnel, each quadrant is 7.85 ft (2.38m) wide for a total circumference of 31.4 ft (9.5). The exposed tunnel surface is mapped from the top down, with the crown in the center of the mapping sheet and the invert divided in two on either edge of the mapping sheet.

Daily field mapping sheets were generated at a scale of 1 in = 10 ft. An example daily field mapping sheet is shown on Figure 9.

In general, mapped features included contacts between distinct sandstone, siltstone, and shale bedrock units; bedding and structural discontinuities in the bedrock units; areas of groundwater flow; areas exhibiting distinct weathering; and unusual features that could provide information on the potential rock mass behavior.

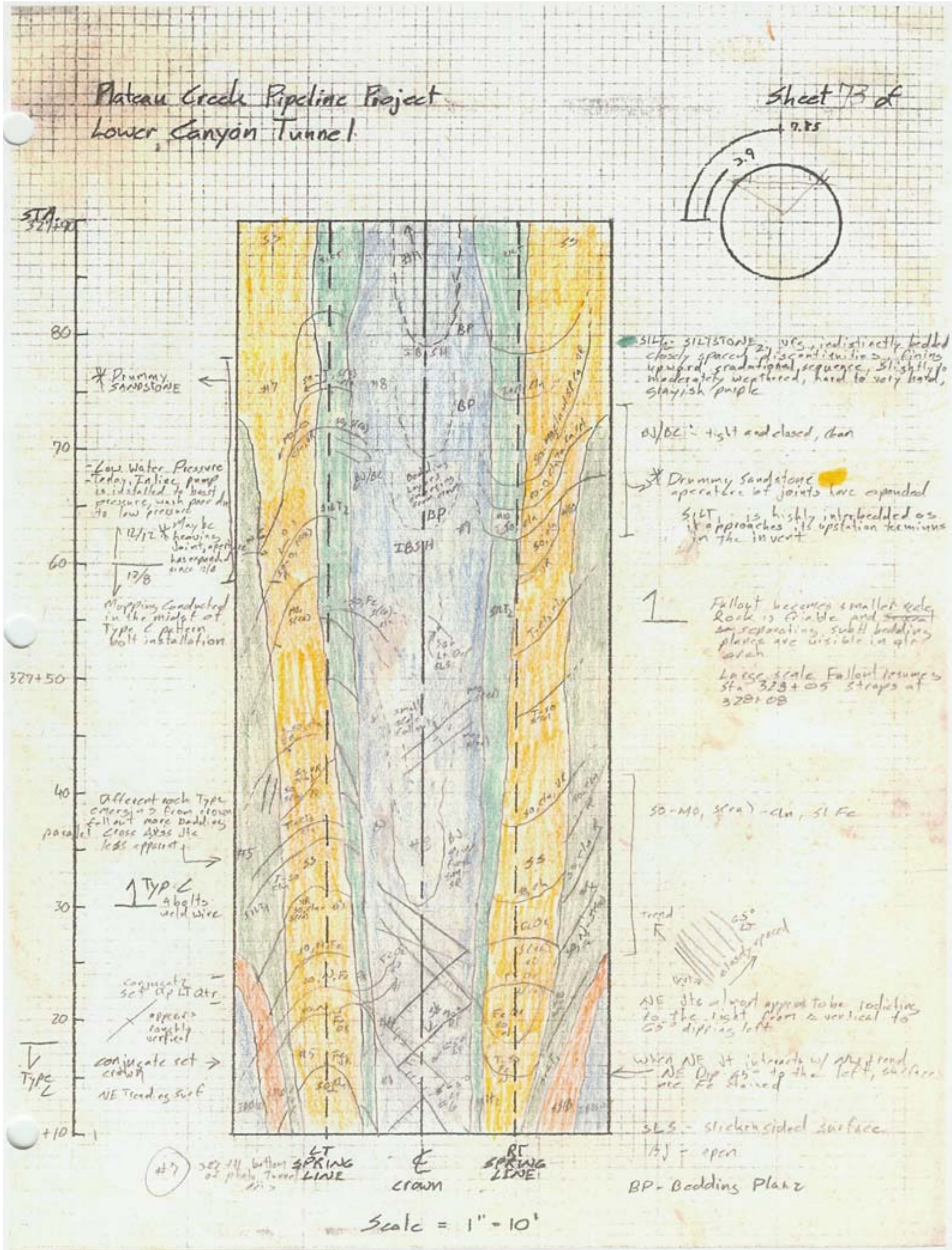

Figure 9. Example Tunnel Mapping Field Log.

Contractual Documents

As is becoming typical of underground construction, a baseline geotechnical report was prepared to establish baseline geotechnical conditions for bidding purposes (GEI, 1999). In addition, the contract documents called for a dispute review board (DRB). DRB's are also becoming typical of underground construction. The DRB members were selected by the owner and contractor. The CRB convened twice during the construction to observe the construction techniques used by the contractor, the conditions encountered in the tunnels, and to discuss any issues that had arose. No dispute issues were brought before the DRB.

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MAP OF CRITICAL LANDSLIDES IN COLORADO - A YEAR 2002 REVIEW

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Key Terms: Colorado, geologic hazards, landslides, debris flows, rockfall, Priority List, wildfire-burn areas, cooperative projects

ABSTRACT

This map and table present an update and revision of the Priority List of Critical Landslides that was an essential part of the Colorado Landslide Mitigation Plan which was adopted by Colorado by Executive Order in 1989. The intent is to identify Colorado communities, areas, and facilities most at risk from landslides and debris flows. The rationale for including a priority list in the plan was to provide an action list of manageable size where scarce staff and funding resources from a variety of sources would yield the greatest benefits. This concept has proven effective over the past 15 years with significant progress in evaluation and/or mitigation being made in more than one-half of the areas. Funding and other substantial contributions have been provided by more than 20 state, federal, local, academic, and private organizations.

INTRODUCTION

The Colorado Landslide Hazard Mitigation Plan was published in 1988 as Colorado Geological Survey (CGS) Bulletin 48 (Jochim et al, 1988). It was written by authors from the CGS, the Colorado Division of Disaster Emergency Services (now Office of Emergency Management), and the University of Colorado Center for Community Development and Design. The plan was adopted by the State of Colorado and was cited for implementation along with Flood Hazard and Wildfire Hazard plans in the Governor's 1989 Executive Order that created the Colorado Natural Hazards Mitigation Council (CNHMC). The CNHMC has a standing committee on Geologic Hazards and a subcommittee on Landslides.

One of the tasks done by CGS for Bulletin 48 was preparation of a list of Colorado's communities, areas and facilities most at risk from landslides. That list consisted of 49 locations believed at that time to pose the most serious landslide threats (Jochim et al, 1988, pages 37-44). The list was prepared using "landslide" in its broadest sense, which included debris flow and rockfall areas. Hazard areas in which the predominant hazard was debris flows were listed separately in recognition of the fact that they nearly always occur in association with stream courses and their depositional fan areas. Both of these conventions are retained in the new priority list herein.

The rationale for including this priority list as an essential element of the Colorado Landslide Hazard Mitigation Plan was to provide an action list of manageable size in which scarce staff and funding resources from a variety of sources would yield the greatest benefits. This concept has proven effective over the past 15 years with significant progress in evaluation and/or mitigation being made in more than one-half of the areas. Funding and other substantial contributions have been provided by more than 20 state, federal, local, academic, and private organizations.

The year 2002 review and priority list was done as part of an update of the 1988 Colorado Landslide Mitigation Plan in cooperation with the Colorado Office of Emergency Management. Our charge is to review and revise the action list as needed. The resulting report was published as Colorado Geological Survey Open File Report No. 03-16 (Rogers, 2003). With permission of the CGS, it has been adapted for inclusion in this volume on Engineering Geology in Colorado for the 2003 Association of Engineering Geologists annual meeting at Vail, Colorado.

Changes in the year 2002 priority list include additions, deletions and reorganization. This results in some very similar adjacent areas of the older list being grouped together and some local hazard areas being incorporated into larger hazard corridors. The previous list was not arranged by hazard severity, but alphabetically by the county in which the hazard was located. This led to confusion for some users and to breaking of logical hazard corridors at county lines into two areas. The revised list presented herein groups the hazards by relative severity into three tiers, as described below. Within each tier, the hazards are arranged alphabetically by county. Also for each tier, hazard areas predominated by debris flows will be listed separately from all other landslides. Each hazard area or corridor is given a number to readily relate the text to the index map (Plate 1).

Description of Tiers

1. **Tier One** listings are serious cases needing immediate or ongoing action or attention because of the severity of potential impacts.
2. **Tier Two** listings are very significant but less severe; or where adequate information and/or some mitigation is in place; or where current development pressures are less extreme.
3. **Tier Three** listings are similar to tier two, but with less-severe consequences or primarily local impact.

   Several listings from the 1988 priority list have been deleted, while others have been incorporated into a larger hazard area or corridor. Deleted areas do not appear on the index map (Plate 1). Those that were regrouped are shown only as part of the newly expanded hazard-area listing. For a more complete description of the individual landslide areas, and the changes reflected in the 2002 priority list, CGS Open File Report No. 03-16 (Rogers, 2003) should be consulted.

DISCUSSION

The Colorado Landslide Mitigation Plan has been in place for more than 15 years. Part of that document - the Priority List of Critical Landslides - has been reviewed, updated and revised. This report presents the resulting year 2002 priority list. Of the 49 areas listed in 1988, thirty have remained intact on the new list. Six areas were deleted, either because of effective mitigation or additional information that downgraded the perceived hazard. Eight listings from the 1988 priority list were "doubled up" with an adjacent hazard area to form four larger hazard areas. Four very small hazard areas on the 1988 priority list are now included in two extensive hazard-corridor areas. Finally, nine entirely new areas have been added, based on new landslide activity and information.

The alphabetical order used for the 1988 priority list has been replaced by a system of three tiers, which are based on estimates of the severity of the hazard and extent or magnitude of potential impacts. Although the priority landslide areas of the year 2002 priority list are numbered sequentially from 1 through 46, there is no intent to indicate relative severity except for the tier designation.

Creation and maintenance of a priority landslide list is of necessity an ongoing process. New landslide events occur and new hazard studies are completed, and our knowledge of natural and human-derived influences evolves. The extent and intensity of our use of the land continues to increase to accommodate Colorado's rapid population growth, with accompanying needs for residential, infrastructure, and commercial development. All of these factors place more people and facilities in potentially hazardous areas, creating new hazard situations. On the other hand, some listed hazard areas may be effectively mitigated, and additional knowledge of other previously listed areas may allow them to be removed or downgraded. For these reasons, we conclude that the landslide priority list should be thoroughly reviewed and revised as needed, but at no greater than 10-year intervals.

The year 2002 priority list, as well as all previous lists (e.g., Rogers, 1986; Jochim et al, 1988; Colorado Water Conservation Board, 1985), were derived from the collective knowledge and experience of the Colorado Geological Survey staff during the past 35 years. During that time, we had extensive contact with other geologists and engineers, and participated in numerous cooperative landslide projects with the Colorado Department of Transportation (CDOT), U.S. Geological Survey (USGS), U.S. Bureau of Reclamation (USBR), local governments, and professional consulting organizations. We have also worked with staff and graduate students at many academic institutions to encourage and support geologic hazard studies. This combination of institutional knowledge and valuable input from our peers has provided us with the background to identify and spotlight 46 critical landslide areas for special attention. It is our hope that this list will continue to be useful in focusing scarce staff and funding resources from many sources: state, federal, and local government as well as academic and private sources, to evaluate and mitigate Colorado's most severe landslide hazards.

ACKNOWLEDGEMENTS

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In a broader context, stalwart support and encouragement for the challenging task of maintaining a landslide program throughout my tenure at the CGS came from colleagues and managers at the Colorado Geological Survey, the Colorado Department of Transportation, the U.S. Geological Survey, and the Colorado Office of Emergency Management. Many other federal, state, local government, academic, and private industry groups and individuals contributed assistance and support in a variety of ways. The importance of this networking with the many other dedicated workers in the field of landslide studies cannot be over emphasized. With the ebb and flow of official/public interest and funding for landslide studies and mitigation, the peer-group contacts helped to maintain a subsistence-level program during the inevitable lean years. For such contributions by a very large number of unnamed individuals, I also express my sincere thanks.

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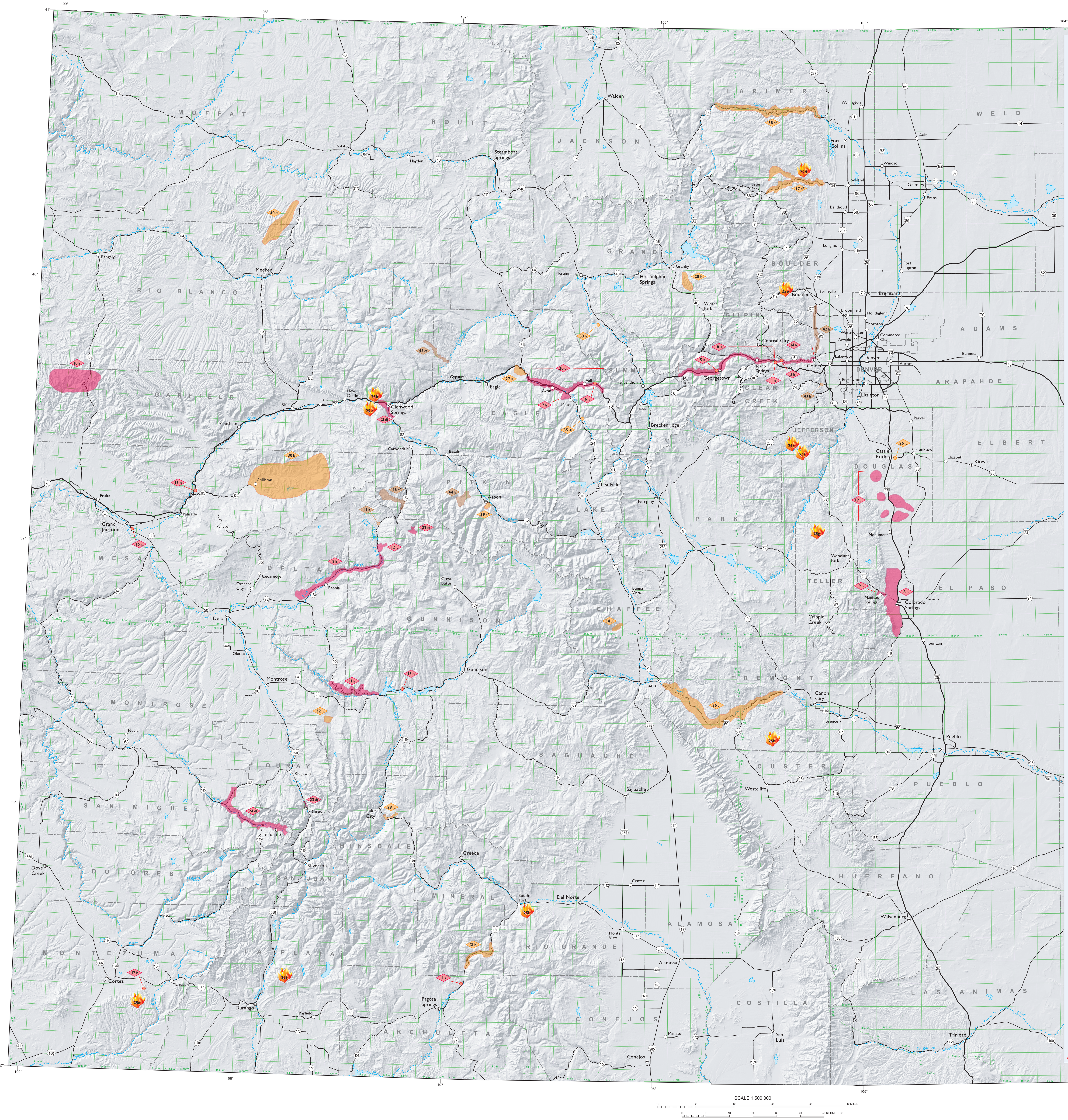
COLORADO GEOLOGICAL SURVEY
DIVISION OF MINERALS AND GEOLOGY
DEPARTMENT OF NATURAL RESOURCES
DENVER, COLORADO

CRITICAL LANDSLIDES OF COLORADO

By William P. Rogers



LANDSLIDE LOCATIONS AND CAPSULE DESCRIPTIONS

Tier One Landslide/Rockfall Areas

No.	Name of Feature	Location and Center Township	Description and Impacts, References
1 ls	San Juan River (Jackson Mountain), landslide	Archuleta County, 0.5 mile below confluence of East Fork and West Fork. T. 36 N., R. 1 W.	Active landslide affecting U.S. Hwy. 160 and utility lines. It is known to have been active since about 1970 and has severed the highway several times since then, requiring closures. **44**
2 ls	North Fork of Gunnison River, landslide areas	Delta and Gunnison Counties, North Fork corridor from Hotchkiss to the Paonia Reservoir. T. 13 S., R. 90 W.	Extremely active landslides along entire corridor, severe rockfall hazard on west side of Paonia Reservoir. Landslides affect Colo. Hwy.133, D&RGW Railroad (now Union Pacific), mine and irrigation facilities of the valley. **27, 44**
3 ls	Clear Creek Forks (Junction), rockslide	Clear Creek County, south side of Clear Creek Canyon on U.S. Hwy. 6 near junction with Colo. Hwy. 119. T. 3 S., R. 72 W.	Active rockslide showing intermittent slow movement since 1940s. Highway damage is ongoing and blockage of Clear Creek is possible. **44,48**
4 ls	Floyd Hill grade, rock and debris slide area	Clear Creek County, on I-70 east of U.S. Hwy. 6 junction near bottom of Floyd Hill grade. T. 4 S., R. 72 W.	Large intermittently active rock and debris slide affecting I-70. Blockage of Clear Creek is possible. **44**
5 ls	Georgetown Incline, rockfall area	Clear Creek County, on west side of I-70 and extending from Georgetown to Silver Plume. T. 4 S., R. 74 W.	Very severe rockfall hazard from steep cut slopes and natural slopes. Causes damage, high maintenance and closures of westbound I-70 lanes. Hazard to travelling public including vehicle damage, injuries and occasional fatalities. Major mitigation was begun in 2002. **1, 44**
6 ls	Booth Creek, rockfall area	Town of Vail, Eagle County, on debris fan of Booth Creek. T. 5 S., R. 80 W.	Very severe rockfall hazard to affected residents. Partially mitigated by ditch and berm barrier in 1990, western part of area in the condominium area urgently needs barrier construction. **44, 58, 71**
7 ls	Dowds Junction, landslides	Eagle County, at junction of I-70 and U.S. Hwy. 24 on southwest side of both highways. T. 5 S., R. 81 W.	Complex of four large old landslides, activated in lower regions (toes) by highway construction. Continuing, sporadic damage to both I-70 and U.S. Hwy. 24. The Dowds No. 1 slide shows slow, large-scale ongoing movement west of the eastbound I-70 Eagle River bridge. This entire area is a case study in the 1988 Colorado Landslide Mitigation Plan. **18, 22, 26, 36, 44, 53**
8 ls	City of Colorado Springs, reactivated old landslides	City of Colorado Springs, El Paso County, various locations between I-25 corridor and the mountain front. T. 14 S., R. 67 W.	Extensive areas of marginally stable hill slopes and old landslides. Modification under urbanization triggers sporadic landslides, damaging or destroying residences and infrastructure. **7, 14, 72**
9 ls	Manitou Springs town site, rockfall, debris flow and flash flood area	Manitou Springs town site and vicinity, El Paso County. T. 14 S., R. 67 W.	Much of the existing town site and adjacent growth areas are subject to intermittent rockfall, landslide, debris flow, and flash flooding activity. **44**
10 ls	Douglas Pass/Baxter Pass region, landslide and debris flow areas	Garfield County, a very broad area including the passes and approaches on both sides of the Colorado River/White River divide. T. 5 S., R. 102 W.	This is an extremely active landslide and debris flow area that affects Colo. Hwy. 139, a county road, and critical energy-related infrastructure facilities. **44, 59, 62**
11 ls	Black Mesa, landslide, earthflow and rockfall corridor	Gunnison and Montrose counties, along Colo. Hwy. 92, from vicinity of Blue Mesa Reservoir dam westerly along the north rim of the Black Canyon of the Gunnison River. T. 49 N., R. 5 W.	This highway corridor is periodically subject to landslides, earthflows, and rockfall. The new status of the Black Canyon as a National Park will greatly increase the need for safe and adequate access to the North Rim sites along the Colo. Hwy. 92 alignment. **1, 40, 44**
12 ls	East Muddy Creek, landslides and earthflows	Gunnison County, on east side of Muddy Creek and Colo. Hwy. 133, starting just upstream of Paonia Reservoir and extending about 2.5 miles north. T. 12 S., R. 89 W.	This is currently a very active landslide area that is a reactivated older landslide complex. Monitoring and surface observations show continuing movement with the south slide being the most threatening. Disruption of Colo. Hwy. 133 and blockage of the flow of Muddy Creek appear to be impending. **2, 44, 60, 63, 64, 66**
13 ls	Red Creek, landslide	Gunnison County, on U.S. Hwy. 50 and north shore of the Blue Mesa Reservoir near Red Creek. T. 49 N., R. 3 W.	This is a reactivated, old landslide that extends below the reservoir level and periodically causes extensive damage to the highway. **44, 70**
14 ls	Clear Creek Canyon/US Hwy 6, rockfall corridor	Jefferson and Clear Creek Counties, along U.S. Hwy. 6 corridor in Clear Creek Canyon from near Golden to the junction of U.S. Hwy. 6 and I-70 east of Idaho Springs. T. 3 S., R. 71 W.	This hazard corridor consists of intermittent to nearly continuous rockfall segments that seriously affect safety and maintenance of U.S. Hwy. 6. There is greatly increased traffic and exposure to the public since low stake gambling was initiated in Black Hawk and Central City. **1, 44**
15 ls	DeBeque Canyon (Tunnel), landslide	Mesa County, on south side of I-70 and the Colorado River, within DeBeque Canyon and about 1 mile upstream from the I-70 Beaver Tail Tunnel. T. 10 S., R. 97 W.	This is a complex landslide that had its modern origin in a catastrophic rockslide/landslide early in the 20th century. Currently, it periodically disrupts the I-70 highway. A comprehensive geotechnical study was completed in April, 2000 by CDOT, CGS, Golder Associates, and CSM. **23, 44**
16 ls	Lamplite Park, landslide	Grand Junction, Mesa County, in the Orchard Mesa area adjacent to the Colorado River. T. 1 S., R. 1 W. (Ute Meridian)	This landslide is periodically activated by bluff retreat caused by the Colorado River eroding the bluff base. Ten homes were damaged and removed in the 1980s and three residences remain in the high hazard zone. This site was a case study in the Colorado Landslide Mitigation Plan of 1988. **11, 13, 26, 44**
17 ls	Mesa Verde National Park, access road landslide	Montezuma County, Point Lookout area of Mesa Verde National Park access road. T. 35 N., R. 14 W.	This is a mile-long segment of the main (and only) access road that has been subject to repeated landslides since the Park opened in 1929. The landslides have caused closures and detours that are a frequent and serious detriment to this popular National Park. **20, 39, 44**

Tier One Debris Flow Areas

No.	Name of Feature	Location	Description and Impacts, References
18 df	Clear Creek/I-70 corridor, debris flow and rockfall areas	Clear Creek County, along I-70 from its junction with U.S. Hwy. 6 east of Idaho Springs to the East Portal of Eisenhower Memorial Tunnel. T. 3 S., R. 74 W.	Intermittent debris flow and rockfall areas, including parts of most towns and development clusters. Threatens public and private property, the traveling public and the I-70 roadway. Will become even more important if this route becomes a rapid transit corridor. **1, 9, 10, 14, 33, 44, 51, 56, 73**
19 df	Steep sideslopes of mesas, Douglas County, debris avalanches	Douglas County, near I-25 corridor between Castle Rock and the El Paso County line. T. 10 S., R. 67 W.	Certain steep mesa sideslopes and adjacent footslope areas are subject to debris avalanching and debris flow runout. These events are sporadic but potentially very dangerous. **44, 52**
20 df	Vail and adjacent development corridor, debris flow areas	Eagle County, intermittent to nearly continuous areas from East Vail to Wolcott in the valleys of Gore Creek and the Eagle River. T. 5 S., R. 81 W.	These debris flow hazard areas consist of the debris/alluvial fans of tributary streams as they reach the major valley floors. They are subject to frequent but unpredictable debris flow events. Some areas also subject to snow avalanches. **33, 44**
21 df	Glenwood Springs and vicinity, debris flow areas	Glenwood Springs town site and vicinity, Garfield County. T. 6 S., R. 89 W.	More than 20 steep mountain stream courses enter the narrow valley floors of the Colorado and Roaring Fork rivers in and around Glenwood Springs. The area has been severely impacted throughout its history by damaging debris flows. Underlying older deposits of these debris fan areas are composed of hydrocompactive soils that cause additional potential building hazards. The Glenwood Springs area was a case study in the 1988 Colorado Landslide Mitigation Plan. **19, 26, 30, 32, 44**
22 df	Marble town site and vicinity, debris flows	Marble town site, Gunnison County, on the extensive debris/alluvial fans of Carbonate and Slate Creeks and other smaller creeks of the area. T. 11 S., R. 88 W.	The debris/alluvial fans of this area are subject to frequent and destructive debris flows that have plagued the area throughout its history. Active channels of the fans shift often and most of the remaining structures are quite vulnerable to future flow events. **28, 37, 44, 45, 47**
23 df	Ouray town site and vicinity, debris flows	Ouray town site and adjacent areas, Ouray County. T. 44 N., R. 7 W.	Because of its location in the narrow canyon of the Uncompahgre River, Ouray is located almost entirely on the debris/alluvial fans of Portland, Cascade and Oak Creeks, all of which have been subject to numerous large and destructive debris flows events in historic times. Recent development has occurred on the fan of Sky Rocket Creek, and other debris fan areas may be considered for future development. Some structural mitigation is in place but may not be adequate to protect lives and property from future large debris flow events. **25, 44**
24 df	Telluride and San Miguel River corridor, debris flow and rockfall areas	Telluride town site and vicinity, San Miguel County, including the San Miguel River Valley corridor west to Placerville. T. 43 N., R. 10 W.	This entire area is subject to frequent debris flows from the numerous, steep tributary streams that form the debris fans of the valley fringe. Rockfall is also a serious hazard, especially from the cliffs of the north valley wall. Present and future residential areas and infrastructure are vulnerable at many locations. In August 2001, more than 20 debris flows caused havoc along the entire Colo. Hwy. 145 corridor of this area. Roads were engulfed and damaged and vehicles were swept into the San Miguel River. **14, 35, 44, 65**
25	New and recent wildfire burn areas, potential for debris flows, rockfalls, and extreme erosion	Various locations throughout forest and brush land of Colorado. These include the following recent wildfire areas: **25a** Mesa Verde National Park, Montezuma County. T. 35 N., R. 14 W., 2000 **25b** Storm King Mountain, Garfield County. T. 5 S., R. 89 W., 1994 **25c** Hi Meadows, Jefferson County. T. 7 S., R. 71 W., 2000 **25d** Bobcat, Larimer County. T. 6 N., R. 71 W., 2000 **25e** Black Tiger, Boulder County. T. 1 N., R. 71 W., 1989 **25f** Buffalo Creek, Jefferson County. T. 8 S., R. 71 W., 1996 **25g** Hayman, Teller and Douglas Counties. T. 11 S., R. 70 W., 2002 **25h** Iron Mountain, Fremont County. T. 20 S., R. 72 W., 2002 **25i** Million, Rio Grande County. T. 39 N., R. 3 E., 2002 **25j** Missionary Ridge, La Plata County. T. 36 N., R. 8 W., 2002 **25k** Coal Seam, Garfield County. T. 6 S., R. 90 W., 2002	Loss of vegetative cover and water repellent soils resulting from wildfire burns can vastly increase the sediment and debris flow potential of watersheds. Recent examples include Black Tiger Gulch, Boulder County; Storm King Mountain, Garfield County; and Buffalo Creek in Jefferson County. In all of these areas, there were extremely severe and dangerous debris flow and flash flood events following wildfires. The hazards tend to diminish through time as the burn areas become revegetated. As this report was in final preparation several major wildfire burn areas of 2002 were added. **3, 4, 5, 6, 16, 24, 29, 44**

Tier Two Landslide/Rockfall Areas

No.	Name of Feature	Location	Description and Impacts, References
26 ls	Town of Castle Rock, rockfall areas	Castle Rock town site, Douglas County, residential areas at base of the low cliffs in the northeast part of town. T. 8 S., R. 67 W.	Large slabs of caprock can become detached and move both as rockslide and rockfall during seasons of high slope moisture. One serious event that occurred in 1981 was detected and mitigated without property damage or injury. **14, 44, 52**
27 ls	I-70 corridor and adjacent private lands near Wolcott, landslide	Eagle County, I-70 roadway and private lands from Bellyache Ridge northeast to I-70 near Wolcott. T. 4 S., R. 83 W.	Very large, old landslide complex that has been re-activated in part by construction of I-70. Adjacent private lands to the southwest are marginally stable and suitable only for selective and prudent development. **42, 44**
28 ls	Fraser Canyon (Amtrak), landslide areas	Grand County, in the Fraser River Canyon between Tabernash and Granby. T. 1 N., R. 76 W.	This area is subject to landslides, debris flows and rockfall. A very small lanslide in 1985 severed the embankment and tracks, resulting in a major derailment of the Amtrak passenger train. Mitigation has been done and warning devices have been installed. This area typifies many of the hazardous canyons of Colorado with vulnerable railroad routes. **12, 44, 49**
29 ls	Slumgullion landslide/earthflow	Hinsdale County, near Lake City on the Lake Fork of the Gunnison River. T. 43 N., R. 4 W.	This very large and famous landslide/earthflow formed the natural dam that created Lake San Cristobal. The upper part is very active and is a long-range threat to Colo. Hwy. 149. Extensively studied by the U.S. Geological Survey and is not believed to pose short-term hazards. **17, 21, 38, 44, 50, 69**
30 ls	Vega Reservoir and Buzzard Creek area, landslides and earthflows	Mesa County, in the area surrounding Vega Reservoir and Buzzard Creek to the vicinity of Collbran. T. 9 S., R. 93 W.	These areas have historically been very prone to landslides. The most recent widespread events were in the middle 1980s. At risk are public roads that provide access to residential, recreational and energy production activities. **44, 54, 55**
31 ls	Wolf Creek Pass area, landslides, debris flows and rockfall	Mineral County, Colo. Hwy. 160 corridor on both sides of Wolf Creek Pass. T. 37 N., R. 1 E.	This has historically been an area of unstable slopes, rockfall and landslides. CDOT has corrected many of these problems in the past 15 years, improving the reliability and safety of the highway for the traveling public. **37, 44**
32 ls	Wells Basin, landslide	Montrose County, on west side of the Cimarron River Valley five miles south of the Cimarron community. T. 47 N., R. 7 W.	The Wells Basin topographic feature is created by a very large old landslide. A very active landslide that is a small part of the older landslide has moved about a thousand feet, displacing Montrose County Rd. P77 and the irrigation ditch that is adjacent. Efforts to mitigate have had very limited success to date. **44**
33 ls	Green Mountain Reservoir area, old landslide complex	Summit County, on the south shore of Green Mountain Reservoir, including all or most of the community of Heeney. T. 2 S., R. 80 W.	This is a large, ancient landslide that has been mapped as extending 1.5 miles along the south shore of Green Mountain Reservoir. It probably extends below reservoir levels. New movement during 2002 drought as a result of reservoir draw down is under evaluation by USBR. **41, 44**

Tier Two Debris Flow Areas

No.	Name of Feature	Location	Description and Impacts, References
34 df	Chalk Creek area, debris flows and rockfall	Chaffee County, along Chalk Creek in the vicinity of Mt. Princeton Hot Springs and extending to the base of the Chalk Cliffs to the northwest. T. 15 S., R. 79 W.	This area consists of multiple debris flow fans with numerous shifting distributary channels. Rockfall hazards are present below the Chalk Cliffs. At risk are numerous residential structures and youth summer camp facilities. Little or no mitigation has been done and both existing and future development needs attention. **37, 44**
35 df	Red Cliff town site, debris avalanche and rockfall	Red Cliff town site, Eagle County, north side of town from "high road" to Turkey Creek. T. 6 S., R. 80 W.	This area experienced very severe debris avalanche/debris flow activity in 1984 and 1985. Cooperative efforts by state agencies and Eagle County assisted in structural mitigation that should be monitored for condition and performance. Rockfall hazard has not been evaluated, but may be serious. **34, 44, 61**
36 df	Lower reaches and alluvial fans of Arkansas River tributaries	Fremont County, along U.S. Hwy. 50 and Arkansas River corridor between Salida and Parkdale. T. 48 N., R. 12 E.	U.S. Hwy. 50, Colo. Hwy. 69 and county roads have been flooded periodically with rock, mud, woody debris and flood water, requiring frequent cleanup and roadway repairs after the large events. **44**
37 df	Tributary streams to Big Thompson River, debris flows and flash flooding	Larimer County, Big Thompson Canyon/U.S. Hwy. 34 corridor between Loveland and Estes Park. T. 5 N., R. 71 W.	This is the area of the catastrophic flood in Big Thompson Canyon of 1976. Much of the damage and loss of life at that time was from debris flows and debris slides that accompanied the mountain-torrent flooding. Most destroyed homes were not rebuilt, and more conservative land use regulations have decreased but not eliminated hazards of this area. **31, 44, 57**
38 df	Poudre River corridor, debris flows, landslides	Larimer County, Colo. Hwy. 14 corridor in Poudre Valley between Fort Collins and Rustic. T. 8 N., R. 72 W.	This corridor contains numerous residential and commercial clusters and campgrounds as well as Colo. Hwy. 14. All are vulnerable to mountain-torrent flooding, isolated debris flows, debris or rockslides. A large rockslide in 1999 caused a six-week road closure. **44**
39 df	Aspen Mountain Ski Area and vicinity, debris slides and debris flows	Pitkin County, in and adjacent to the ski area and in resedential areas on debris fans or foot slopes. T. 10 S., R. 85 W.	Several slope failures and/or debris flows have occurred in the 1980s and 1990s. Considerable mitigation has been done by the ski area and other property owners. The area still has potential for further problems and should be closely monitored by owners and city/county officials. **8, 44**
40 df	Devils Hole Gulch / Wilson Creek area, debris flows, landslides and extreme erosion	Rio Blanco and Moffatt Counties, starting approximately 10 miles NNW from Meeker and extending along Devils Gulch, through the Wilson Creek Oil Field and northeast along the axis of Wilson Creek to its confluence with Taylor Creek. T. 3 N., R. 94 W.	This area experienced extreme debris flow, landslide, and erosion activity in the middle 1980s. County roads, oil field roads, and electrical transmission lines were severely damaged. The facilities have been repaired or relocated, but future damage of a similar nature is probable. **44**

Tier Three Landslide/Rockfall Areas

No.	Name of Feature	Location	Description and Impacts, References
41 ls	West side of McClure Pass along Lee Creek, landslides and earthflows	Gunnison County, on the west side of Lee Creek on Colo. Hwy. 133 about 5 miles past the summit of McClure Pass. T. 11 S., R. 89 W.	These landslides originate in weak and unstable soil and rock that are upslope from Colo. Hwy. 133. They displace the roadway and become shallow earthflows on lower slopes below the highway. These were very serious during the period 1970-1986. Since that time, additional maintenance and repair have not been excessive. **44, 46**
42 ls	Golden to Boulder along CO Hwy 93, landslides and earthflows	Jefferson and Boulder Counties, along Colo. Hwy. 93 from Golden to Marshall. T. 2 S., R. 70 W.	Slopes adjacent to the Colo. Hwy. 93 corridor are mostly composed of weak claystone bedrock and derived soils. Earlier routes were plagued by severe landsliding. During the last 15 years, much of the highway has been widened, realigned or relocated with generally excellent results. **44**
43 ls	Morrison town water plant, landslide	Morrison town site, Jefferson County, in the southwest part of town near Bear Creek. T. 5 S., R. 70 W.	This landslide became active in 1985. Landslide material upslope from the water plant began to move downslope, threatening the facility. Most of the landslide mass was removed at the time, and no subsequent problems have been reported. **44**
44 ls	Snowmass Village and vicinity, landslides, slumps, and earthflows	Snowmass Village and vicinity, Pitkin County, residential areas and ski lift tower locations. T. 10 S., R. 86 W.	In the 1970s and 1980s there were numerous landslide problems both in developing residential areas and with structures on the ski slopes. Local governments, ski area managers, and developers and their consultants are aware of the problems, and appear to have been successful in dealing with them during the past 12 years. **44**

Tier Three Debris Flow Areas

No.	Name of Feature	Location	Description and Impacts, References
45 df	Sweetwater Creek area, debris flows	Garfield and Eagle Counties, along Sweetwater Creek north of Dotsero. T. 3 S., R. 87 W.	This is a remote area that is still sparsely developed with recreational and residential facilities, especially near Sweetwater Lake. Numerous debris flows that required roadway maintenance were reported and investigated by CGS in 1985 . There have been no subsequent reports of serious problems. **44**
46 df	Dutch Creek, Coal Creek, and Redstone area, debris flows, backwater flooding, and erosion	Redstone town site, Pitkin County, channels of Coal Creek and tributaries from the Crystal River at Redstone to the Coal Creek basin coal mining area. T. 10 S., R. 88 W.	Tributaries of the upper basin continue to experience frequent, large debris flows, but the coal mines are abandoned and under reclamation. The remaining hazard is at Redstone where the debris from the upper basin obstructs the confluence of Coal Creek and the Crystal River, causing backwater flooding and erosion that affects the town site and Colo. Hwy. 133. Town, county, and CDOT maintenance staff deal with these problems each year during spring runoff. **44**

EXPLANATION

Landslide or Rockfall Area or Corridor

12 ls

Tier One landslide—denoted by magenta/red color, listings are serious cases needing immediate or ongoing action or attention because of the severity of potential impacts. Identification number of landslide or rockfall area or corridor inside diamond. Approximate location of smaller landslide or rockfall area indicated by circle, approximate extent of larger landslide or rockfall area indicated by polygon.

30 ls

Tier Two landslide—denoted by orange color, cases are very significant but less severe; or where adequate information and or some mitigation is in place; or where current development pressures are less extreme. Identification number of landslide or rockfall area or corridor inside diamond. Approximate location of smaller landslide or rockfall area indicated by circle, approximate extent of larger landslide or rockfall area indicated by polygon.

44 ls

Tier Three landslide—denoted by brown color, cases are similar to Tier Two, but with less severe consequences or primarily local impact. Identification number of landslide or rockfall or corridor inside diamond. Approximate location of smaller landslide or rockfall area indicated by circle, approximate extent of larger landslide or rockfall area indicated by polygon.

Debris Flow Area or Corridor

22 df

Tier One debris flow—denoted by magenta/red color, listings are serious cases needing immediate or ongoing action or attention because of the severity of potential impacts. Identification number of debris flow area or corridor inside diamond. Approximate location of smaller debris flow area indicated by circle, approximate extent of larger debris flow area indicated by polygon.

25c

Tier One wildfire burn—approximate location denoted by flame symbol, listings are serious cases needing immediate or ongoing action or attention because of the severity of potential impacts. Identification number of wildfire burn area inside flame.

35 df

Tier Two debris flow—denoted by orange color, cases are very significant but less severe; or where adequate information and or some mitigation is in place; or where current development pressures are less extreme. Identification number of debris flow or corridor inside diamond. Approximate location of smaller debris flow area indicated by circle, approximate extent of larger debris flow area indicated by polygon.

46 df

Tier Three debris flow—denoted by brown color, cases are similar to Tier Two, but with less severe consequences or primarily local impact. Identification number of debris flow or corridor inside diamond. Approximate location of smaller debris flow area indicated by circle, approximate extent of larger debris flow area indicated by polygon.

Data Sources

State Boundary modified from Colorado Department of Transportaion

Counties and Highways from Colorado Department of Transportaion

Township and Range from Tobin International, Ltd.

100 meter Digital Elevation Model from U.S. Geological Survey

Digital Cartography by Jason C. Wilson

* Landslide (ls) or Debris Flow (df) designates only the predominant hazard process. More complete information and description of areas are contained in CGS Open–File Report 03–16.

LANDSLIDE-SUSCEPTIBILITY MAPPING IN COLORADO SPRINGS, EL PASO COUNTY, COLORADO

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Key Terms: landslide susceptibility, mapping, GIS, land-use planning

ABSTRACT

The City of Colorado Springs lies at the boundary between the Great Plains and the southern Rocky Mountains. The western part of the city occupies a series of dissected foothills underlain by weak Cretaceous claystones, predominantly the Pierre Shale, which are prone to instability. Early development avoided the foothills; however, city growth has caused these areas to become increasingly developed since the 1980s. This expansion is fueled by premium land prices for infill and view lots. Many of these areas have been previously mapped as landslide deposits. The land-use issue relating to development in landslide susceptible areas is becoming a key issue in planning and land management in many areas across the United States. A recent estimate by the United States Geological Survey stated that 25 to 50 deaths and damage exceeding $2 billion occur annually in the U.S. due to landslides (Spiker & Gori, 2000).

During the wet spring seasons of 1995 and 1999, ground movements occurred in Colorado Springs, many of which became well publicized in the local media. Slope modification and lawn irrigation, along with natural processes, appear to have been a factor in several episodes. Many of the landslide occurrences were in developed areas that were insufficiently investigated for potential geologic hazards and were not analyzed for slope stability. Homeowners were unaware of the risk to their properties until ground movement damaged or destroyed their homes.

Landslides remain a controversial topic in Colorado Springs, and public awareness of landslides has grown. However, the City, its business community, and the majority of the voting populace are strongly protective of property rights for new and existing land uses. This means that area-specific landslide investigations and recommendations must be based on solid scientific principles and engineering assumptions in order to justify the resulting, sometimes unfavorable, land-use decisions.

The Colorado Geological Survey (CGS) provides the City with landslide-hazard assessments as part of its land-use review function, and has recently completed a GIS-based map of select landslides and landslide-susceptible areas for the City (White & Wait, in press). The landslide-susceptibility map is based on the geology, topography, geomorphology, hydrology, and landslide history of the Colorado Springs area. A landslide inventory for the City was compiled and used to evaluate common geologic conditions and factors that may lead to slope failure. This information was then used to determine areas that may be susceptible to landslides. The map is scheduled to be published by CGS in 2003.

INTRODUCTION

Landslides are one of the most costly natural hazards in the United States, threatening every state in the United States. Landslides are the result of the force of gravity acting on a slope where ground conditions are sufficiently weak that soil or rock materials begin to move or slide downhill. These movements can range from very rapid, singular events, such as rockfalls or debris flows, to very slow ground movements that are only perceptible over months or years. Structures not designed for earth movements generally do not survive landslide movements. The tremendous earth forces will shift, shear, crack, move, or bury buildings. Once initiated, landslide movements often continue until the damage is such that the structure is completely destroyed or becomes unsafe, requiring condemnation.

Colorado Springs lies at the boundary between the Great Plains and the southern Rocky Mountains about 65 miles (104.6 km) south of Denver (Figure 1). The western part of the city consists of a series of foothills and pediment mesas underlain by weak, overconsolidated Cretaceous claystones that are prone to landslides (Brooker & Peck, 1993). Several areas in the city have experienced various degrees of damage from landslides during the 1990s. The springs of 1995 and 1999 corresponded with wet winters and long-duration spring rainstorms, climatic conditions that resulted in higher frequencies of ground movements, many of which became well-publicized in the local media. Human-caused factors, such as slope modification and lawn irrigation, appear to have played a part in several of these episodes.

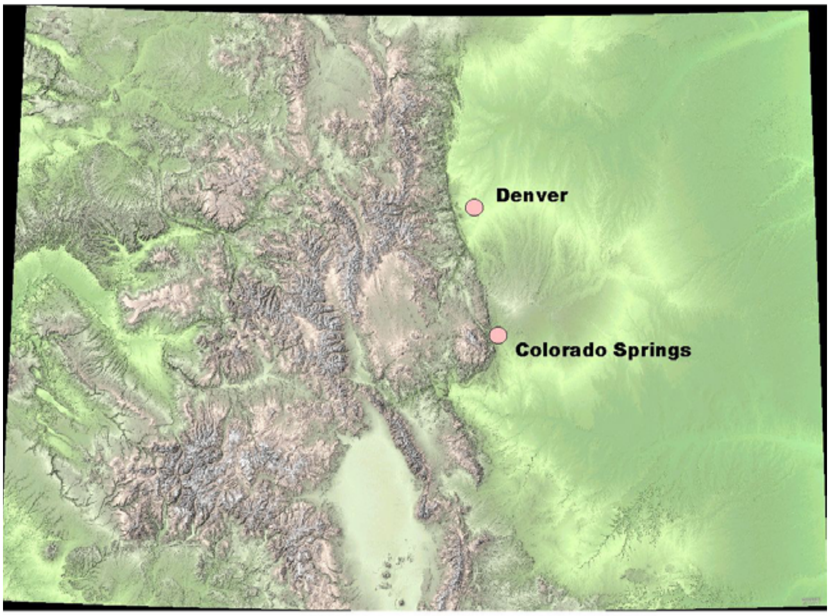


Figure 1. Colorado Springs location map.

The flooding and landsliding in 1999 caused widespread and significant damage. Subsequently, a Presidential Disaster Declaration was issued for Colorado Springs and El Paso County that made federal relief available. Part of the federal response was that the Federal Emergency Management Agency (FEMA) provided Colorado Springs with over $4.5 million in funds, under the Unmet Needs Program, to acquire landslide-affected properties. The mapping described in this paper was a follow-up project to the emergency FEMA program.

For the purposes of this paper, the authors have chosen to use the term "landslide" to describe ground movements involving weak claystone and shale rock rotational and translational slides and slumps that may involve unconsolidated soils and can evolve into earth flows, as defined by Cruden and Varnes (1996). This paper describes the methods used to create the landslide-susceptibility map for Colorado Springs that shows areas of landslide susceptibility and outlines of known landslides within the city limits. Its intent is to provide overlay map coverage that will aid ongoing city planning, allow for general public information disclosure and dissemination, and prompt a level of future geological and geotechnical investigation that is appropriate for the hazards and potential risks present. Other forms of ground movement, including shallow creep, ground subsidence, swelling and heave from expansive soils and bedrock, and rapid forms of mass movement such as rockfall, rock avalanches, and debris flows were intentionally not included in the scope of this project. These types of geologic hazards, while still significant in Colorado Springs, were not mapped for this project.

The mapping methodology applied a modified heuristic or qualitative method described by Souters and van Westen (1996) using a basic inventory of landslides, published geologic maps of the area (Scott & Wobus, 1973; Cochran, 1977 a-e; Trimble & Machette, 1979; Carroll & Crawford, 2000; Thorson et al. 2001; and Rowley et al. in press) and non-published geologic maps, known engineering characteristics of bedrock and derived soils, and digital geologic and topographic information. Previous landslide-susceptibility studies and projects (Ahmad & McCalpin, 1999; Wegmann & Walsh, 2001) and earlier Colorado Springs-specific land-use/zoning and hazard reports (Hill, 1974; Gruntfest & Huber, 1985) were reviewed for this project. Other data were derived from various GIS data sources, photo-interpretations, and field checking by trained engineering geologists.

BACKGROUND GEOLOGY OF THE COLORADO SPRINGS AREA

Colorado Springs straddles the High Plains section of the Great Plains and southern Rocky Mountains physiographic provinces. East of Interstate 25, the city lies on rolling hills of the High Plains. West of Interstate 25, the city rises in elevation towards Cheyenne Mountain, Pikes Peak, and the Rampart Range, which border the city to the west. Within the city limits, ground elevations range from 5720 ft (1743 m) to 9212 ft (2808 m) above sea level, an elevation change of 3492 ft (1064 m). Figure 2 shows the generalized geology and topography of the Colorado Springs area.

Two thrust-fault zones in the Colorado Springs area, the Rampart Range and Ute Pass faults, mark the eastern edge of the Cenozoic (Laramide Orogeny) uplifting of the Front Range. These faults have shown continued movements into the Quaternary Period (Widmann et al. 2002). Complex geologic structures are found where the faults converge around Garden of the Gods and Manitou Springs. Tilted, steeply-dipping, even vertical and overturned rock formations formed by uplift and by thrust-fault drag are found in the late Paleozoic and Mesozoic rock formations in the foothills of the mountain front. Younger Cretaceous and Cenozoic sediments become gradually less tilted to the east (Figure 3). These overconsolidated Cretaceous claystone and

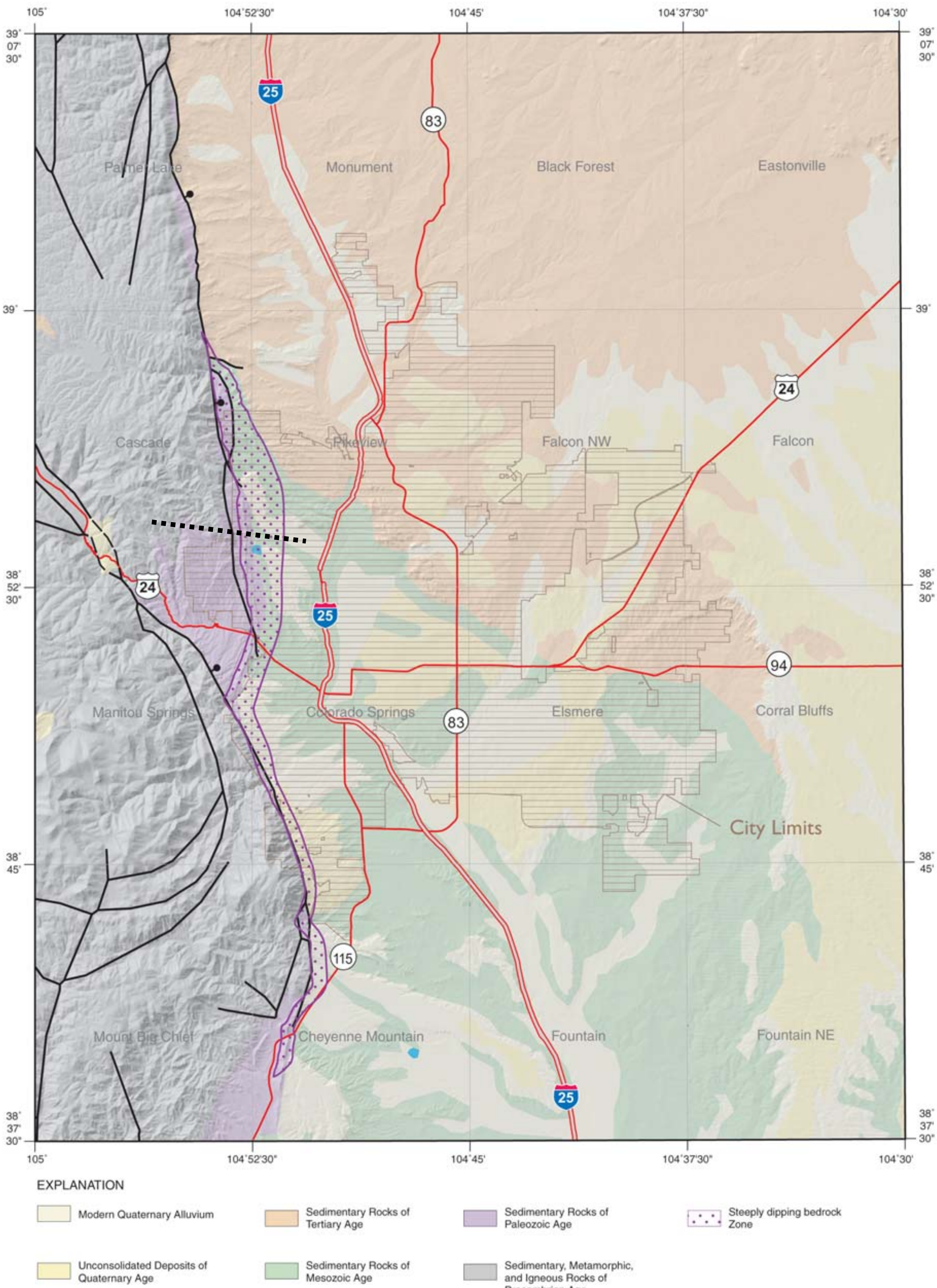


Figure 2. Generalized geologic and digital elevation map of the Colorado Springs area. Dotted line indicates approximate cross section shown in Figure 3.

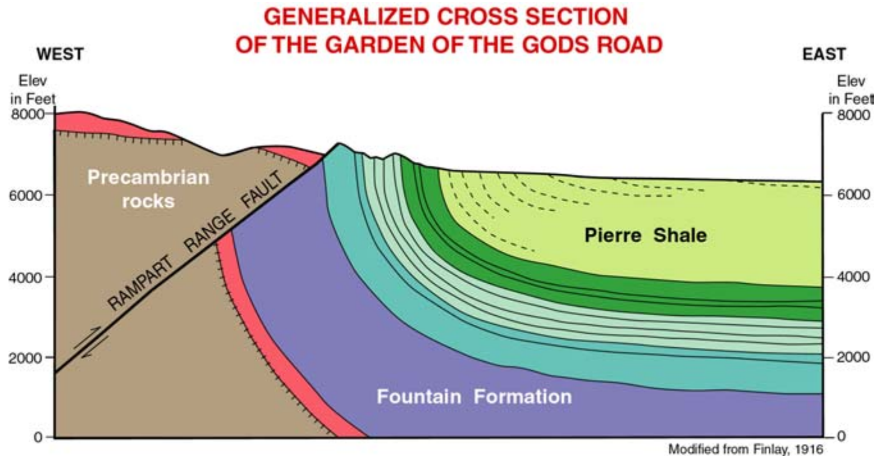


Figure 3. Generalized cross section through Colorado Springs showing bedrock dip along the Garden of the Gods road alignment (Himmelreich and Noe, 1999). Approximate location of section is shown on Figure 2.

shale-rich formations, such as the Pierre Shale, dominate much of the landslide-susceptible terrain along the foothills west of Interstate 25.

Pleistocene erosion and deposition processes eroded basement rock from the Front Range, moved sediment from the mountains, and deposited sand and gravel on pediment surfaces that cap the claystone bedrock. Late Pleistocene and Holocene erosional downcutting has incised these pediments and underlying bedrock, forming high mesas from the pediment remnants. Deposition of alluvial and debris-flow sediment continues to occur along the mountain front. All of these processes have combined to create the modern foothills seen today. The steep slopes along the margins of the mesas are prone to mass-wasting processes, such as landsliding, which is a natural erosional process for many of the clay-rich colluvial and bedrock slopes.

Landslide Hazards

The main factors that affect whether a landslide will occur are topography, geology, and hydrology. These factors influence the inherent internal strength of the rock or soil materials that comprise the slope and, accordingly, the slope stability. Very strong, massive rock can sustain a vertical slope without failure. Very weak rock and soil materials can only hold a low or moderate slope without experiencing shear failure and lateral ground movements. Landslide analysis is commonly done by limit-equilibrium methods, a comparison of driving forces versus resisting forces. Landslides are triggered when some critical slope-stability threshold is met and the driving forces exceed the resisting forces. This may occur when the internal strength is lowered as a result of natural processes (e.g., precipitation, weathering, or erosion) or from human influences (e.g., water introduction or adverse ground modification). Ground modifications that contribute to unstable slope conditions include ground removal or loss of lateral support (where the slope base (mass) is removed, decreasing the resisting forces), increased pore-water pressures, and/or the addition of weight, or loading that increases the driving forces near the top of a slope.

Weak rock masses, and the soils derived from them, are generally clay-rich materials. In the Colorado Springs area, several sedimentary formations contain these weak bedrock materials. Derived products include in-situ weathered and disturbed claystones, and residual and colluvial clay soil deposits. Overwhelmingly, the bedrock formations are overconsolidated clay shales deposited in Cretaceous mid-continent seas, with pronounced weaknesses (low or residual shear strengths) along bedding planes (Erskine, 1973; Eversoll, 1991; Brooker & Peck, 1993). Areas in Colorado Springs where these formations occur may be susceptible to landslides if sufficient slope grades exist.

Surface expressions of large-scale and/or more geologically recent landslides can be identified by aerial-photo reconnaissance and field checking. However, small rotational and translational slumps are often unrecognizable by these means, as they often are covered with colluvial soils washed in from above. These types of buried, or "stealth" slides are generally not recognized in drill borings and can only be identified from observations in road cuts, trenches, and deeper excavations. Geologists working in the Colorado Springs area (Noe, 1996; White & Wait, in press; Himmelreich, pers. comm.) have seen evidence of buried landslides in several parts of Colorado Springs within the mapped landslide susceptible areas while observing trench and foundation excavations. This suggests that many more landslides exist that have no surface expression.

Brooker and Peck (1993) illustrated the difficulty in analyzing stability, or even determining failure surfaces for overconsolidated Cretaceous clay shales. Landslides that glide along bedding planes can involve extremely thin (only millimeters in thickness) shear zones that are impossible to detect in normal geotechnical auger drilling and difficult to detect in core samples from rock-core drilling and open excavations. These bedding planes can be nearly horizontal. Bedding-plane shear deformation can exist without escarpments at the surface. Where long-term incision (erosion) and lateral unloading have occurred on overconsolidated shale slopes, those slopes should be considered as having bedding shears near the slope base, and should be analyzed at residual rock-and-soil strength parameters. Such slopes are very sensitive to disturbance, either from natural means or human influences.

Landslide History in Colorado Springs

Landslide hazards and related risks in certain areas of Colorado Springs have been known by geologists for nearly three decades, although most residents in those areas are not aware of the hazards that may impact them. Landslide locations were mapped by Scott and Wobus (1973) and Cochran (1977a-e) prior to the development of some of the more problematic areas. The area was mapped again in 1979 by Trimble and Machette; this map was mostly a recompilation of Scott and Wobus (1973) at a smaller scale.

New landslides and reactivations of older, existing landslides occurred during the wet springs of 1995 and 1999. Ground movements impacted several neighborhoods west of I-25 and many homes were destroyed or condemned. Though all of these locations lie within the mapped landslide-susceptible areas, many of those neighborhoods had no previous history of landslide activity, and homeowners had no knowledge of the hazard to which they were exposed.

This landslide mapping was conducted in partial response to the 1990s landslide events. The GIS database serves to provide the City planners with a tool that can be used to determine a level of study required for areas that may be prone to landslides. The published paper maps are available to the public as a means of understanding areas that may be at risk. While much of this information may already exist, it was not compiled or readily available for public use. The results of this study provide a city-wide scientific prospective of landslide susceptible areas.

MAPPING METHODOLOGY

Mapping for this project was done primarily in a digital GIS-based format with data compiled qualitatively based on historic information, geomorphology, geology, topography, and observed water conditions. Colorado Spring Utilities (CSU), using their Facilities Information Management System (FIMS) data, provided the initial digital project data, including city and park boundaries, photogrammetric 2-ft contours, street centerlines, and high-resolution ortho-rectified air photos from 1995 and 1998. Base data coverages compiled by the City Planning Senior GIS Analyst from the FIMS data included a 5-ft pixel digital elevation model (DEM), slope gradient grid, and slope aspect grid. CGS provided digital geologic maps of the Pikeview (Thorson et al. 2001), Colorado Springs (Carroll & Crawford, 2000), and Cheyenne Mountain quadrangles (Rowley et al. in press), geo-referenced scans of USGS geologic maps (Trimble & Machette, 1979), and digitized El Paso County 1041 Geologic Hazards Maps (Cochran 1977a-e). CGS compiled digital data coverages using the above base coverages in ArcView 3.2a at a 1 in = 800 ft (1:9600) scale. The GIS-based data were then incorporated into USGS topographic base maps for publication at a scale of 1 in = 2000 ft (1:24,000). The mapping process used is described below and also shown in Figure 4.

Known Landslide Inventory

Known landslide location data were collected from published maps and reports, consultant's reports from CGS land-use review files, news articles, and independent consultant sources. These locations were field-verified, then digitized in the ArcView project as a landslide-inventory coverage. Areas with a history of past slope stability problems may be prone to future failure and can also indicate factors that contributed to landslide occurrence. Many of these known landslide sites are sensitive to disturbance by human activity and modification. Some of the landslides were located in areas where slope conditions had since changed by grading and slope reduction to the point that they were no longer considered to have stability concerns and were subsequently eliminated from the susceptibility coverage.

Geomorphology

Geomorphic features can indicate the presence of landslide terrain and provide a relative age based on the amount of erosion that has occurred since the slope failure. Landslide landforms are generally obvious geomorphic expressions that disrupt the original profile of the slope. They may include scarps or slope breaks, mounded toe morphology, back-tilted or rotated blocks, side shears and offsets, and other compression or tension features. Classic terrain such as "step-and-bench," "hummocky," or "lobate" features can also indicate landslide deposits. Shifted or offset

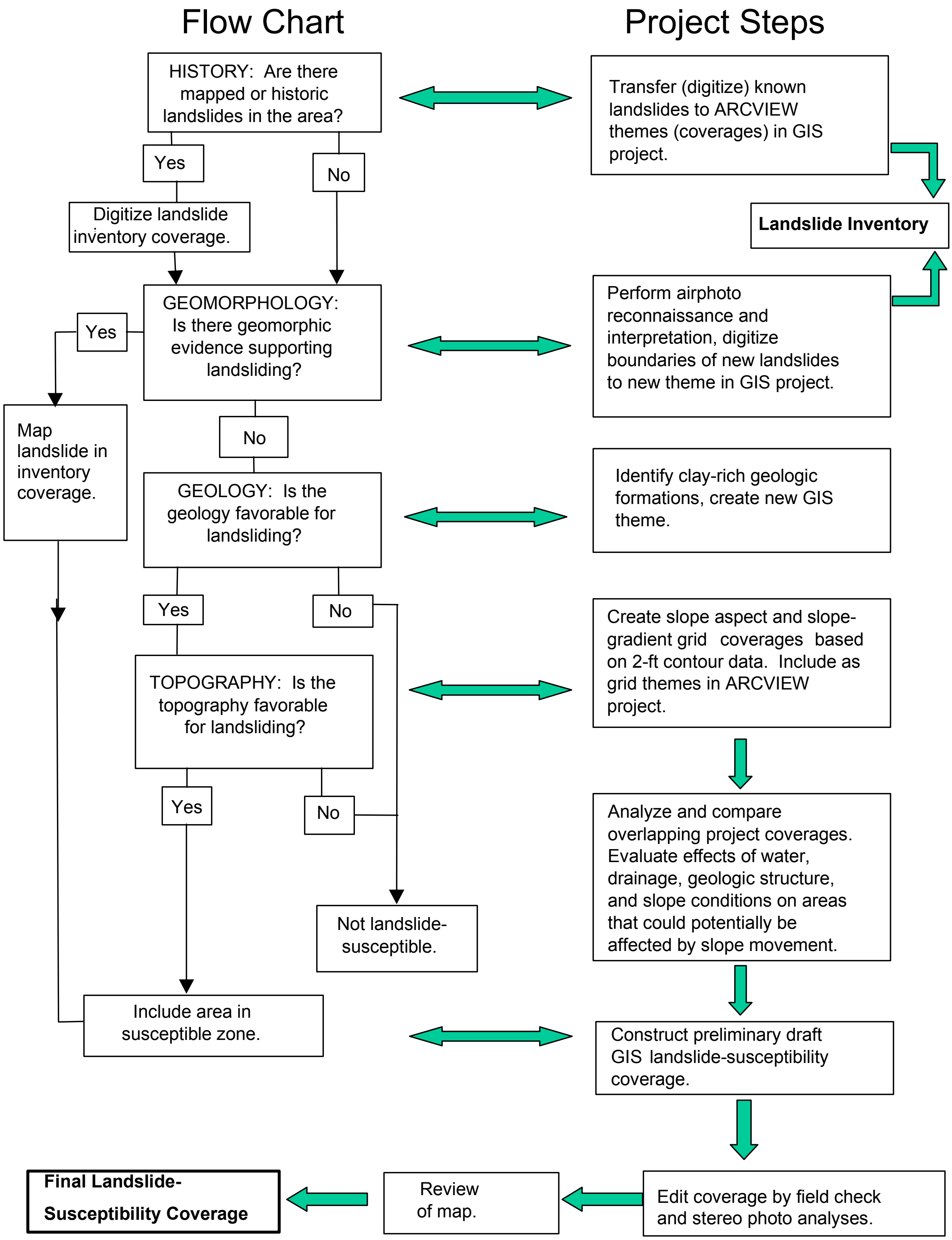


Figure 4. Flow chart showing decisions and related project steps for landslide-susceptibility mapping.

drainage channels often indicate areas that have been affected by landslides. An example of classic geomorphic landslide terrain in Colorado Springs is shown in Figure 5. The DEM and air photos were examined for evidence of landslide deposits. Follow-up fieldwork and stereo aerial photographic analyses either confirmed or eliminated the area as a landslide or landslide susceptible area. Areas that were determined to be landslides were digitized and included in the known landslide-inventory coverage. Areas that were not actual landslides, but appeared to have favorable geologic and topographic characteristics for landslide susceptibility were evaluated for inclusion in the landslide-susceptibility coverage.

Figure 5. Landslide geomorphic features of the Hofstead landslide on the northeast flank of "The Mesa" in Colorado Springs in 1999. Note the "step-and-bench" topography below the head scarp and the cut slope into the old landslide toe. The dashed line indicates the extents of the slide reactivation in 1999. Arrow shows slide direction. Also note the back-tilting of the closest home in the toe. That home, and others, were demolished subsequent to this photo. Photo by M. Squire, OEM.

Geology

Geology, including rock type and geologic structure, have a major influence on slope stability. Clay-rich formations or members are often associated with landslide-prone areas, particularly the Pierre Shale (Erskine, 1973; Eversoll, 1991; Himmelreich, 1996). Overconsolidated claystone, interbedded weak bentonite seams derived from volcanic ash falls in Cretaceous time, existing failure surfaces, and weathering zones all exhibit adverse engineering properties associated with weak rock or soil: low peak shear strength and strength values lowered further to residual levels on slopes or bedding planes that have previously failed.

Geologic structure, such as the orientation of bedding and jointing or fracturing, can also adversely affect slope stability. Bedding planes and other discontinuity surfaces have much lower shear strengths than the intact rock. If the dip direction is the same as the slope direction, large translational sheet-like failures can occur – especially if the bedding dip angle is less than the slope angle. The ground slope will then cross the bedding planes and "daylight" them, resulting in instability if the shear strength of the bedding plane is low enough. Jointed or fractured rock allows water to infiltrate, thus increasing pore pressures and accelerating weathering. This can be a particular problem in the steeply-dipping bedrock area in the western portion of Colorado Springs (Himmelreich & Noe, 1999). In the field, these areas can sometimes be identified by the presence of seeps or vegetation changes along slopes. As areas with these characteristics were identified, they were further evaluated to determine whether slope conditions were also favorable for landsliding.

Particular attention was given to clay-rich bedrock units, which have historically been prone to instability in steeper slope terrain in the Colorado Springs area (Scott & Wobus, 1973; Cochran, 1977; Himmelreich, 1996; Noe & White, unpubl.). These clay-rich formations or members include the Glen Eyrie Shale Member of the Fountain Formation, the Benton Group (Graneros Shale, Greenhorn Limestone, and Carlile Shales), the Pierre Shale, the upper part of the Laramie Formation, and the lower part of the Dawson Formation in the Colorado Springs area. Areas where these units are present were identified in the GIS project and evaluated for slope conditions for landslide susceptibility.

Topography and Slope Aspect

Topography is also a major factor in landslide susceptibility. As slope gradients increase, gravitational driving forces increase and the factors of safety decrease. In the Colorado Springs area, slopes with grades greater than 12% (7˚) and having geologic factors favorable for landsliding (such as colluvial slopes underlain by Pierre Shale) were considered for inclusion in the landslide-susceptibility coverage. Twelve percent (7˚) was used as a criterion based on residual shear-strength properties that can be found in the Pierre Shale in this area (Scott & Wobus, 1973; Hill, 1974; Gruntfest & Huber, 1985). Other clay-rich units are expected to have similar strength characteristics. In addition to having low-strength properties, slopes that are already inherently weak, weathered, or at existing landslides can also fail at lower angles.

Slope aspect (the direction a slope face is oriented) can also be a factor, as discussed above with geologic structure. Northeast-facing slopes are more critical because they will have slopes in the same directions as the predominant dip of bedrock along the mountain front, a situation that is conducive to daylighting of the bedding planes. Aspect also affects the amount of seasonal moisture in the soils of a slope; shaded slopes (north- and east-facing slopes) will remain wetter than sunnier (south- and west-facing) slopes. It should be noted that landslides were recorded for all slope aspects, however.

Other Factors

Basic mechanics of landslides were also used in generation of the susceptibility coverage. Margin zones that fall outside the above-defined slope gradients at the top and bottom of slopes were also included in the susceptibility coverage to account for common behavior of landslides. For example, the head scarp of a landslide on a mesa flank will commonly encroach onto a mesa top. Similarly, an earthflow toe can spread out onto relatively flat terrain beyond the base of the slope.

Susceptible Areas

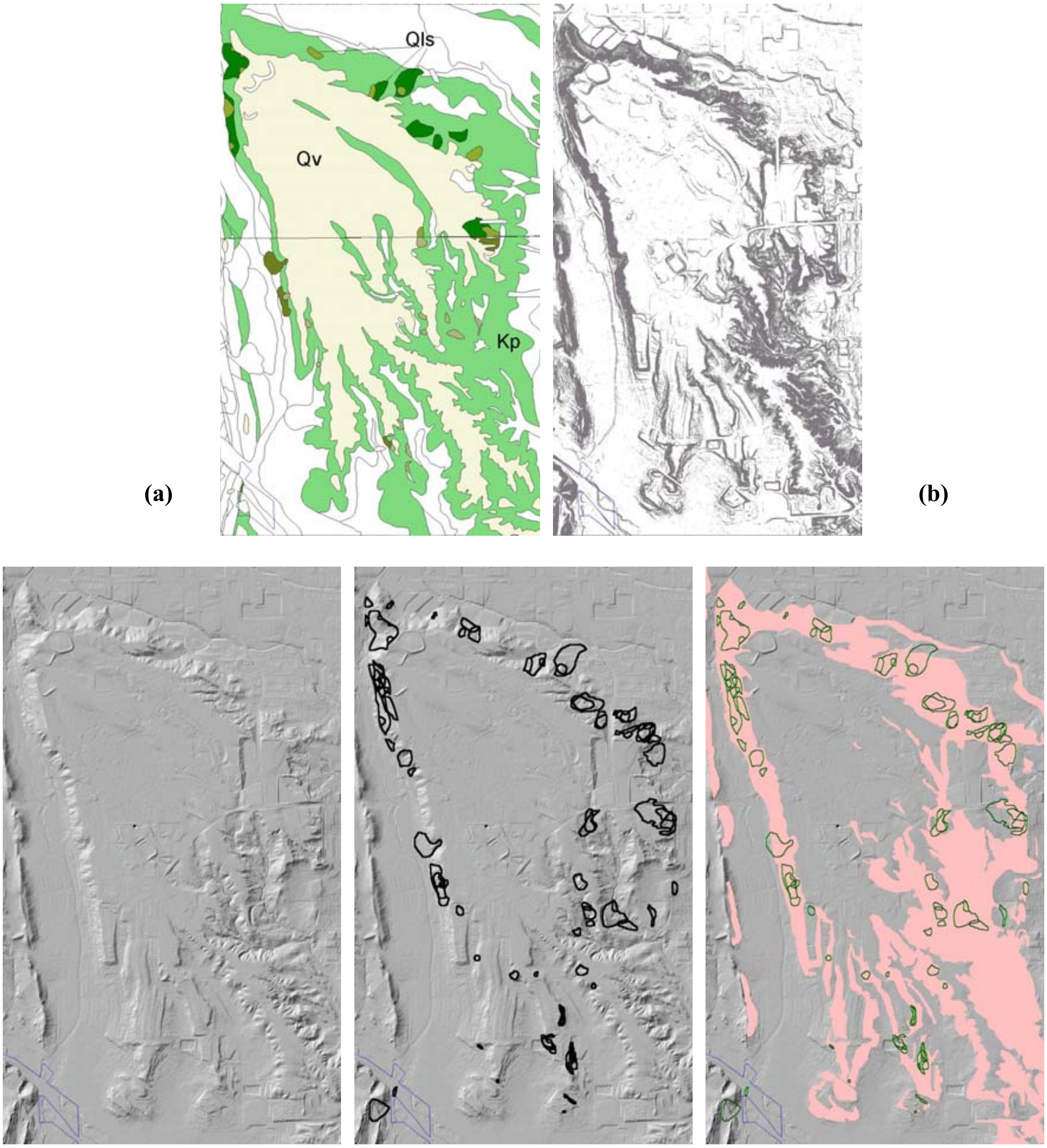
The preliminary susceptibility boundaries were created by tracing a zone that includes areas of the landslide-prone geology and known landslides at defined slope gradients and slope aspects. A more accurate landslide-susceptible boundary was then refined by the inclusions of margin zones (see previous paragraph) and interpretation of stereo aerial photographs. The final susceptibility area was then field checked and revised as necessary. Figure 6 shows how an example area, The Mesa area in northwest Colorado Springs, was mapped using the digital and field data techniques described above.

DISCUSSION AND CONCLUSIONS

The finalized landslide-susceptibility map shows two basic types of information: areas that may be susceptible to landslides and approximate boundaries of known landslides (Figure 7). The landslide-susceptibility map was reproduced for publication at a 1:24,000 scale on USGS topographic base maps, and should be available to the public in 2003. The Colorado Springs Planning Office is currently using a GIS-based version of this map at the scale of 1 in = 800 ft (1:9600) for city planning purposes. Neither the map nor the GIS data are intended to give site-specific information as to slope stability, but rather to serve as a tool for determining areas where slope stability issues may occur.

The areas mapped as being landslide susceptible have geomorphic, geologic, and topographic conditions similar to areas of known landslides, but may not actually experience slope failure from landslides. However, under certain conditions (e.g., heavy precipitation, adverse slope modifications, etc.), slope failure may occur. Areas located within the susceptibility zone should be further evaluated for ground stability and presence of landslide deposits during further development, renovations, ground alterations, road alignments, and residential resale. A thorough geologic investigation and slope-stability analysis should be considered for all real-estate transactions and any new development within the susceptible area. It may be necessary at that time to implement mitigation such as grading, slope drainage, or engineered earth-retaining and/or support systems.

The susceptible-area boundaries are not absolute. A particular area within the susceptible boundary may not be experiencing problems currently, but future conditions involving natural events or human activities such as wetting, cutting the toe, loading a head scarp, and/or continued long-term weathering and weakening of the slope material could result in slope movement in the future. Based on the landslide inventory and an assessment of other factors, the overwhelming majority of the landslide-prone ground in Colorado Springs lies within the mapped susceptible area. It should also be noted that small landslides could occur outside of the susceptible area within the city boundary. Discrete and sporadic clay-rich lenses are known to exist in other geologic formations in the Colorado Springs area where slopes are steep. Poorly designed excavations can also induce slope failure even in high-strength materials. Additionally, other types of slope failure (rockfall, debris flows, etc.) that were not within the scope of this project can occur outside of the landslide-susceptibility zone.



(a) (b)

(c) (d) (e)

Figure 6. Mapping methodology demonstrated by GIS images of “The Mesa” area of Colorado Springs. Images shown are the geology and landslide deposits previously mapped by CGS (a), slope grade grid data (b), DEM based on 2-ft contours (c), finalized landslide inventory (d), and landslide-susceptibility map (susceptible areas shown in pink) (e).

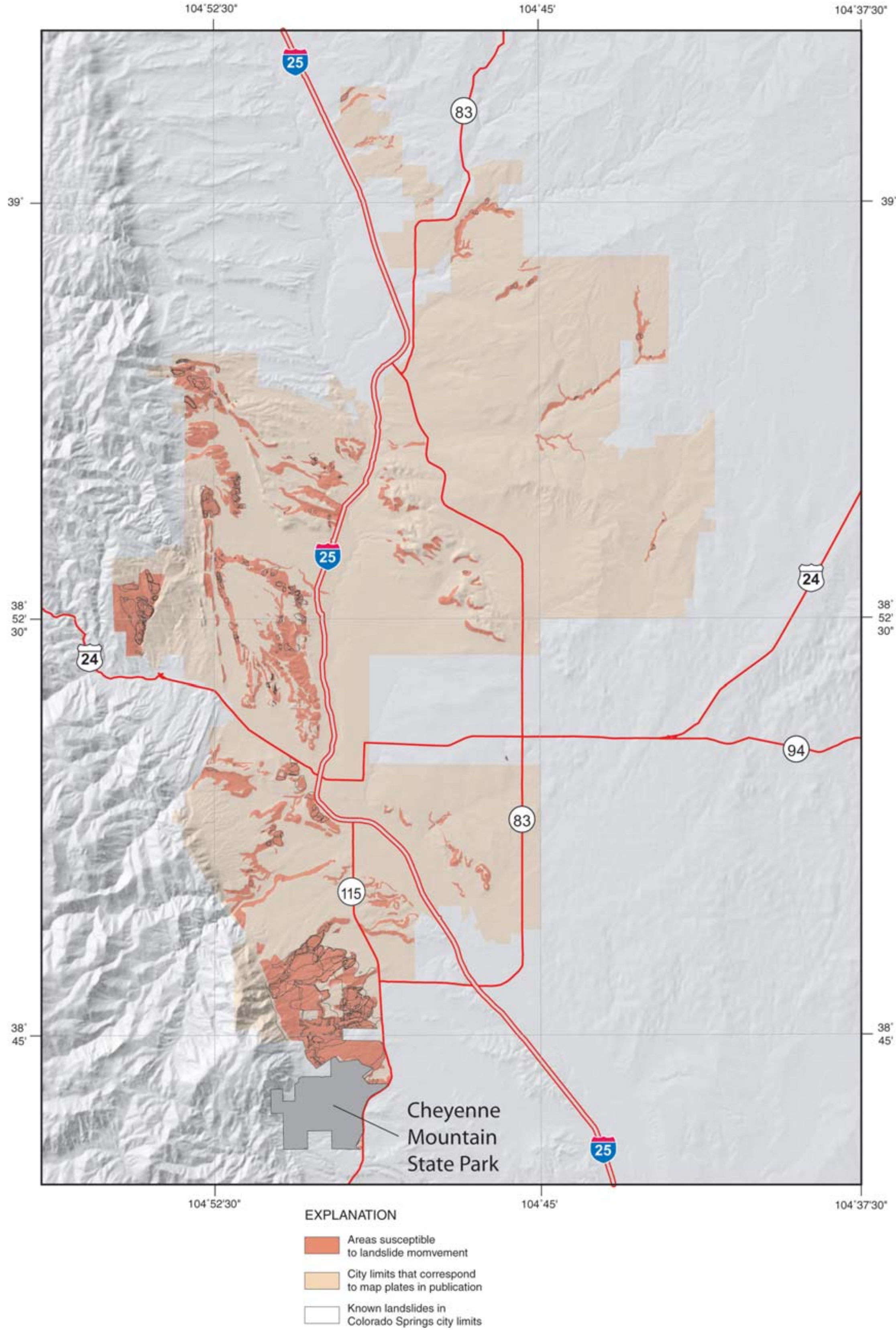


Figure 7. Colorado Springs landslide-susceptibility map.

The approximate boundaries of known landslides were also inventoried and mapped. These boundaries are from a variety of sources: published, consultant's reports obtained during CGS land-use reviews, city documents and reports, newspaper articles, unpublished consultant mapping, aerial-photographic interpretation, and field work. The known landslides include those known to have recent movement and older slides that have not had documented movement. Some of the large-scale landslides (such as one mapped by Scott and Wobus' (1973) near Cheyenne Mountain) probably occurred when climatic conditions were much wetter than at present. This feature is further discussed by Terry, White, and Wait in this publication. Some of the known landslides have since been stabilized by grading or mitigative processes and are not included in the susceptible area. Other large dormant landslides can not be eliminated from the susceptible until they are shown to be stable.

Quantitative approaches, such as deterministic analyses, and statistical and probabilistic risk modeling, were beyond the scope of this project. Because of the uncertainties inherent in geologic assumptions required at most locations (i.e., subsurface geology, structure, and water conditions), no levels of risk assessment were made within the susceptible zone.

ACKNOWLEDGMENTS

This paper is a condensation of the report included in a map series (White & Wait, in press) that is scheduled for publication in 2003 by the CGS. Funding for this study was provided by FEMA (administered by the Colorado Water Conservation Board), the Development Review Unit of the City of Colorado Springs Planning Group, and through the CGS Critical Hazards Program. Colorado Springs Utilities, by licensed agreement, provided digital data (FIMS) and ortho-rectified airphotos used during this project. Copies of the 1966 aerial photography were obtained from the Pikes Peak Library District Photo Archives.

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GEOLOGY, INSTRUMENTATION, AND MOVEMENT HISTORY OF THE DEBEQUE CANYON LANDSLIDE, MESA COUNTY, WEST-CENTRAL COLORADO.

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Key terms: landslide, rockfall, rock mass, photogrammetric analysis, instrumentation, monitoring, water diversion

ABSTRACT

The DeBeque Canyon landslide is located in Mesa County of west-central Colorado, 20 mi (32 km) east of Grand Junction. Interstate 70 near milepost 51 passes through the toe of the landslide. The landslide complex has displaced the south wall of a 500-ft (152-m) deep canyon, incised by the Colorado River into Cretaceous Mesa Verde Group sedimentary strata. The historical record for landslide activity dates to the late 1800s. The last major event in 1998 badly damaged I-70 and, with a renewed awareness of the morphology of the landslide, there was increased concern that future reactivations could be catastrophic in nature and potentially sever the roadway, temporarily dam the Colorado River, and impact the railroad corridor on the opposite side of the canyon. Recent investigations have shown that the landslide complex probably dates to the late Pleistocene and exhibits several mechanisms of both rock- and soil-type slope failures, including rockmass shearing, block gliding and toppling, and translational and rotational soil-type movements. Current monitoring of the landslide activity now reveals ongoing dynamic processes, with movement continuing, even during the drought conditions Colorado is currently (2000-2003) experiencing. These movements forewarn of possible rockmass failures and landslide reactivations in the future.

INTRODUCTION

The DeBeque Canyon landslide, also referred to historically as the "Tunnel Landslide," is a major landslide complex that has historically impacted a major highway, a major railway line, and the Colorado River (Figure 1). The last major re-activation in April 1998 caused Interstate

Figure 1. Oblique aerial view of DeBeque Canyon landslide. View is to the west. Photo by J. White taken in 1999.

70, located at the toe of the landslide, to heave 14 ft (4.3 m) and shift 10 ft (3 m) laterally toward the river. It was only by the rapid response of the Colorado Department of Transportation (CDOT) that Interstate 70, the main east-west highway transportation corridor in Colorado, was not completely closed. Two other large historic landslide movements occurred at the DeBeque Canyon landslide before 1998. Catastrophic movements occurred at the turn of the century (precise date is unknown) when the landslide toe entered the Colorado River and caused flooding that damaged the railroad and structures at the Tunnel work camp on the opposite riverbank. In 1958, during a road-widening project, the landslide toe again heaved the highway 24 ft (7.3 m). A study of the landslide was conducted by Talbott (1969) during the initial planning of the Interstate-70 alignment through the canyon. In subsequent years and after construction of Interstate 70, periodic nuisance-type activity has been reported by CDOT. The landslide is considered a critical landslide in Colorado by the Colorado Geological Survey (CGS) (see Rogers (2003) in this volume), was periodically monitored (Stover, 1985), and included in the Colorado Landslide Hazard Mitigation Plan (Jochim et al. 1988).

In response to the 1998 event and because of the potential for future catastrophic events, the Federal Highway Administration (FHWA) funded a multi-agency task force to investigate, map, analyze, monitor, and design mitigation methods for Interstate 70 where it crosses the toe of the landslide. This task force, administered by CDOT, included the CGS, Golder Associates, Inc., Colorado School of Mines (CSM), and the U.S. Geological Survey (USGS). Geological aspects of the study included a literature search of landslide information, geologic mapping, detailed cross-section construction, descriptions of materials and landslide features, forensic digital photogrammetric analysis, age dating of landslide morphology, and development and installation of a comprehensive instrumentation program. The geologic evaluation provided the basis for conceptual models of the landslide for development of numerical models used in later engineering stages of the investigation conducted by Golder Associates. All of these data were included in a final technical report that was submitted to CDOT in December 2000 (Golder Associates, Inc., 2000).

This paper discusses aspects of the geologic investigation, movement history, and monitoring program of the landslide that were conducted by CGS and CSM. The USGS Central Region Geologic Hazards Team provided important roles in immediate response, monitoring, and landslide dating. By CDOT task order, the CGS and USGS continue to monitor the landslide.

Regional Setting

The landslide is located in DeBeque Canyon near milepost 51 of Interstate 70 on the south side of the Colorado River, about 20 mi (32 km) east of Grand Junction in Mesa County, Colorado (Figure 2). The landslide is on the USGS Cameo 7.5-minute Quadrangle map, centered on the NW ¼ of the NE ¼ in Section 7, Township 10S, Range 97W of the 6th Principle Meridian in west-central Colorado. The climate is semi-arid to arid, with the area receiving about 12 in (30.5 cm) of annual rainfall. Patchy sage and juniper flora is characteristic of the area.

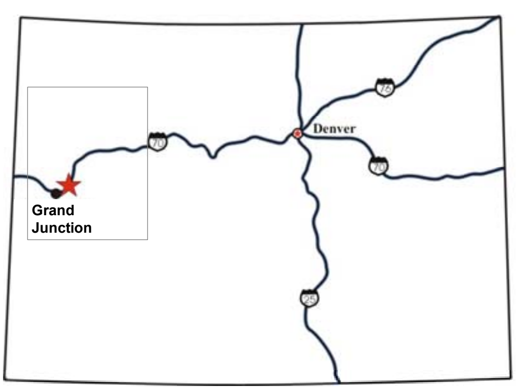


Figure 2. Star denotes landslide location. Box on Colorado location map on left is the approximate area of the map on the right that shows the Piceance Basin area and surface exposures of Mesa Verde Group strata.

DeBeque Canyon is located within the Colorado Plateau physiographic province on the western flank of the Piceance Basin, a Late Cretaceous to early Tertiary structural and depositional basin, the extent of which is defined primarily by the regional Laramide uplifts of the Uncompahgre Plateau along its western flank and the Grand Hogback monocline (White River uplift) along the eastern flank. The landslide is located in the upper Mesa Verde Group; a thick sequence of interbedded sandstone, siltstone, shale, and coal units deposited in non-marine, deltaic/fluvial/coastal plain environments during transgressive and regressive cycles of the Cretaceous intercontinental seaway (Tyler and McMurray, 1995). The Mesa Verde Group is nearly horizontal in the canyon with a gentle regional rise, of about 4 degrees, to the southwest towards the Uncompahgre Uplift. In DeBeque Canyon, the gentle regional dip is interrupted by an unnamed westward-trending, very broad and very gently plunging, anticline and syncline pair (Ellis and Gabaldo, 1989).

The specific bedrock stratigraphy at the landslide includes massive sandstone beds, sequences of thinly interbedded shale, siltstone, and sandstone, as well as a thick problematic shale that overlies a lower, massive, cliff-forming sandstone. Two types of stratigraphic control and description were used for this project. For the geologic map (Map Plate 1), units that were mappable on aerial photographs were assigned letters. A more-detailed and precise subdivision of lithologic units was derived from boring information, surveyed stratigraphic contacts, and detailed profile line surveys. These numbered lithologic units were required for the larger-scale, more detailed, subsurface geology that was incorporated into both limit equilibrium and finite element stability analyses (Golder Associates, 2000). The lettered units used for the map stratigraphy are correlatable to the units described by Talbott (1969).

DeBeque Canyon was created by downcutting of the Colorado River into the relatively resistant sandstones of the Mesa Verde Group. By comparison, the valley is much broader and commonly flanked with Quaternary terrace and pediment remnants where the river flows through the weaker Wasatch Formation upriver at the town of DeBeque, and through the Mancos Shale downriver in the Grand Valley. Gently dipping strata in DeBeque Canyon have the typical morphology of the Colorado Plateau physiographic province where landforms of high, dissected plateaus, large rolling flats mantled with Quaternary loess, and isolated mesas are the norm. Classic cuestas do not appear until the Colorado River exits DeBeque Canyon at Palisade, where regional dip of Mesa Verde strata steepens along the northeast flank of the Uncompahgre Uplift and forms the Book Cliffs.

Geologic mapping methodology

The geologic map of the landslide is shown in Map Plate 1. This map was constructed photogrammetrically, using stereo aerial photography that was flown in 1999. Field mapping was also completed in 1999 and a draft of the map was compiled digitally using Arc-Info and ArcView GIS software. Once the draft map was completed, it was taken to the site, field checked for accuracy, and revised where necessary. Additional edits and revisions were made after the geology field review with task partners and the CDOT task manager in mid-November 1999. Concurrent with the field mapping, wire-line core investigative borings were drilled. Several sites required drill rig mobilization by helicopter.

Subsurface geology of the landslide was interpreted from surface geology, accessible fissures, boring information, and line surveys that were conducted along three profiles. The profiles were oriented so that they would intersect the maximum amount of structural and stratigraphic information. The locations of cross sections A, B, and D were selected for the profiles. Measured survey lines were staked and detailed observations were made of all geologic and morphologic data. Where the lines dropped over the major cliff, technical rock climbers examined the face and held survey prisms at lithologic contacts so their position could be surveyed by electronic distance measurement (EDM) from a base station across the river. Several geologic cross sections shown on the geologic map were developed and subsequently digitized into CAD files for geotechnical analyses. Cross-sections A and B (Figures 3 and 4), highlighted on the geologic map, show the major structures of the landslide complex.

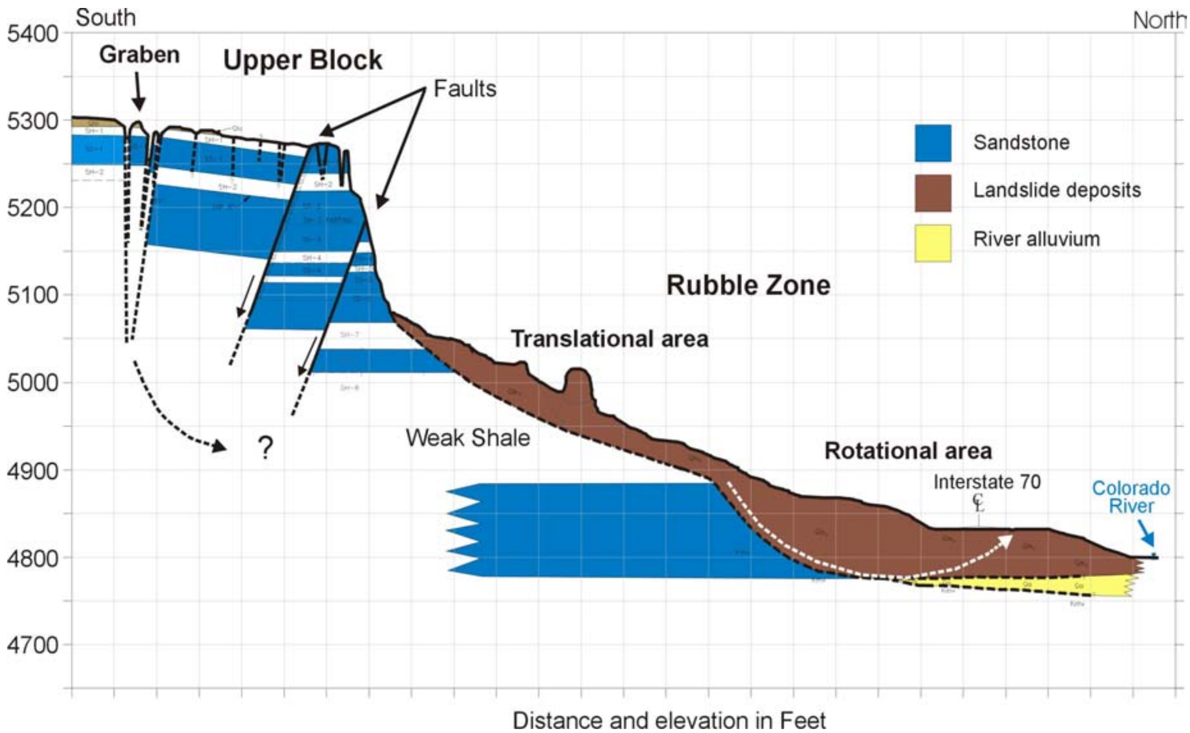


Figure 3. Cross section A. Cross-section line is shown on geologic map plate at end of paper. Certain subsurface sandstone units in Upper Block south of the faults are not shown.

LANDSLIDE MORPHOLOGY

The landslide has complex block glide, toppling, translational, and rotational forms. It covers 36 acres (14.6 ha) and extends from an elevation of 4,800 ft (1,463 m) at the river level to 5,300 ft (1,615 m) at the top of the canyon wall. The landslide was sub-divided into three main divisions based on sliding mechanisms and morphology: the Upper Block, the Rubble Zone, and West Disturbed Block. Figure 5 shows these parts of the landslide complex in oblique view from across the river. Cross-sections A and B (Figures 3 and 4) also show the basic landslide divisions in profile view.

The headscarp of the landslide consists of a main fissure that developed along a major, pre-existing, vertical northeast-trending 4 to 5-ft (1.2 to 1.5-m) wide shear zone. The Laramide-aged shear zone exhibits about 6 vertical feet (1.8 m) of offset (upthrown on the north side) at the

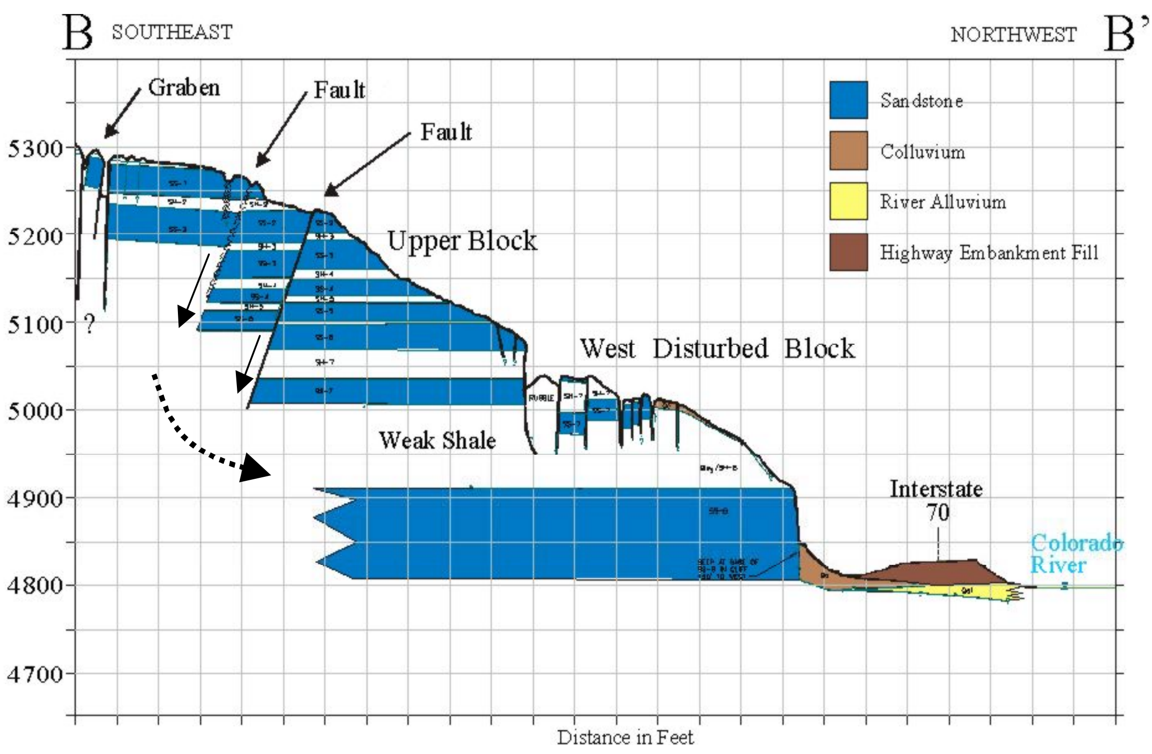


Figure 4. Cross section B. Cross-section line is shown on geologic map plate. Certain subsurface sandstone units are not shown in Upper Block. Note downdropped blocks of sandstone into underlying weak shale at West Disturbed Block and oversteepening of the shale slope that overlies the lower sandstone cliff at road level.

unnamed box canyon on the east side of the landslide. This feature separates the landslide from the intact Mesa Verde Group bedrock that forms the canyon walls, and it parallels a major structural discontinuity pattern in the rocks. Many other fissures and small faults also parallel the main fissure and can be seen on the geologic map plate. A second prominent vertical joint set runs due east and parallels the cliff of the Upper Block above the Rubble Zone. Upon separation and extension along the main fissure, sympathetic joint-defined linear blocks have slumped into the fissure forming a graben morphology. Farther west, as the graben traverses over the west side of the Upper Block, the fissure span closes and, as it curves downslope, narrows and becomes hidden by the colluvial soil cover at the west boundary of the landslide.

The surficial geology in the immediate vicinity of the landslide is predominantly Mesa Verde bedrock with thin colluvial soils on benches, river alluvium on the canyon floor, and a remnant of a mid-Pleistocene terrace on the canyon wall. The top of the canyon is mantled with late-Pleistocene loess. The loess is also fissured on the Upper Block so its deposition predates ground movements.

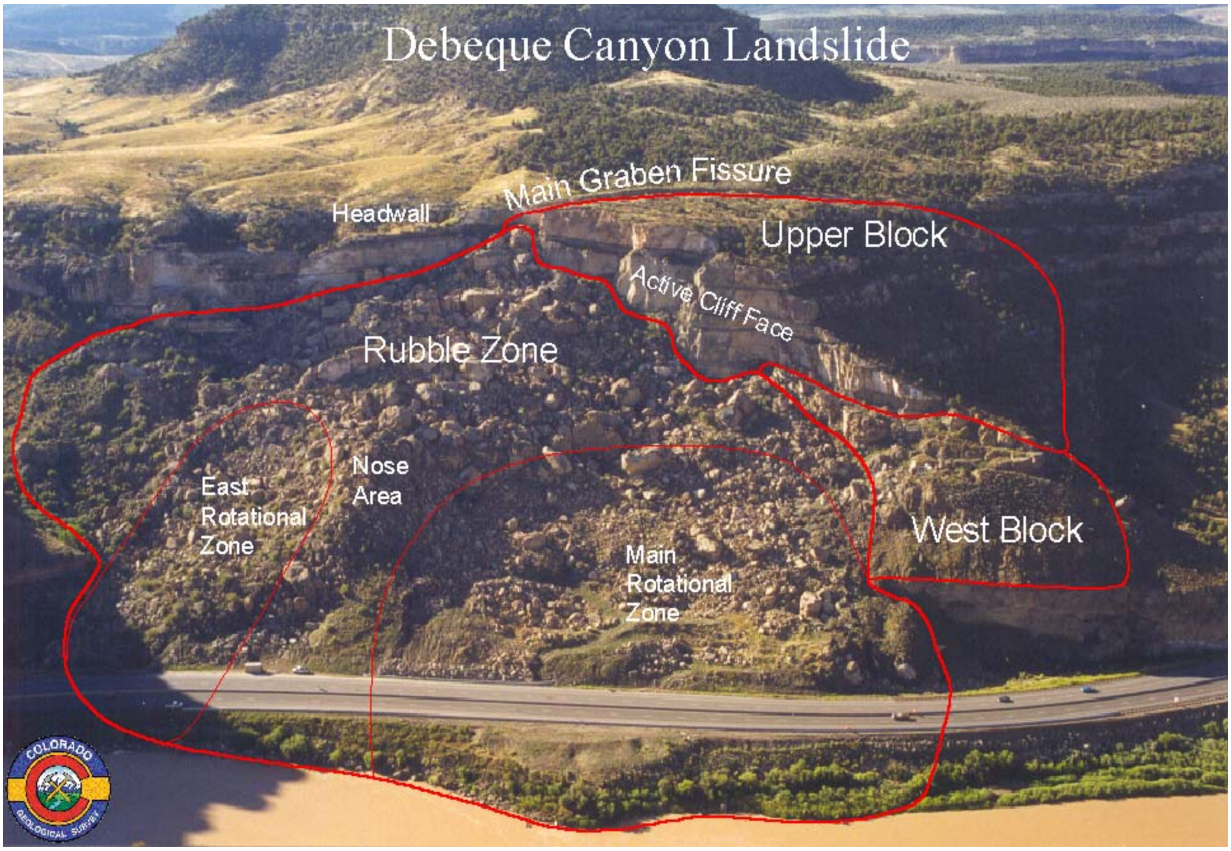


Figure 5. Oblique aerial photograph showing main morphologic divisions of landslide. Photo by J. White taken in 1999. View is to the south.

Upper Block

The Upper Block presents the greatest hazard because of its potential for catastrophic failure and was the major reason for the FHWA-funded study. This fissured, triangular-shaped block is composed of thick sandstone beds and minor thin interbedded shale and continues to move on, or in, a 100-ft (30 m) thick weak claystone bed towards the landslide Rubble Zone and the canyon floor. Profile views of the Upper Block are shown in Figure 3 and 4. If the Upper Block were to fail catastrophically, the highway corridor would likely be blocked and the Colorado River could possibly be partially dammed or diverted and threaten the railroad on the opposite bank. Subsequent failure of a landslide dam could cause serious flooding downriver. The Upper Block has an areal dimension of 1000 x 950 x 680 ft (305 x 290 x 210 m) and is almost 300 ft (90 m) thick. As can be seen in Figure 6, this rock mass has slid to the north towards the river. This displacement has created extensional deformation such as open fissures, depressions, structural offsets (faults), and localized tilting or slumping of the disturbed strata. The Upper Block is the remnant of a much larger mass that previously failed and disintegrated, creating the upper portion of the Rubble Zone. An active cliff face, up to 200 ft (61 m) high, marks the separation

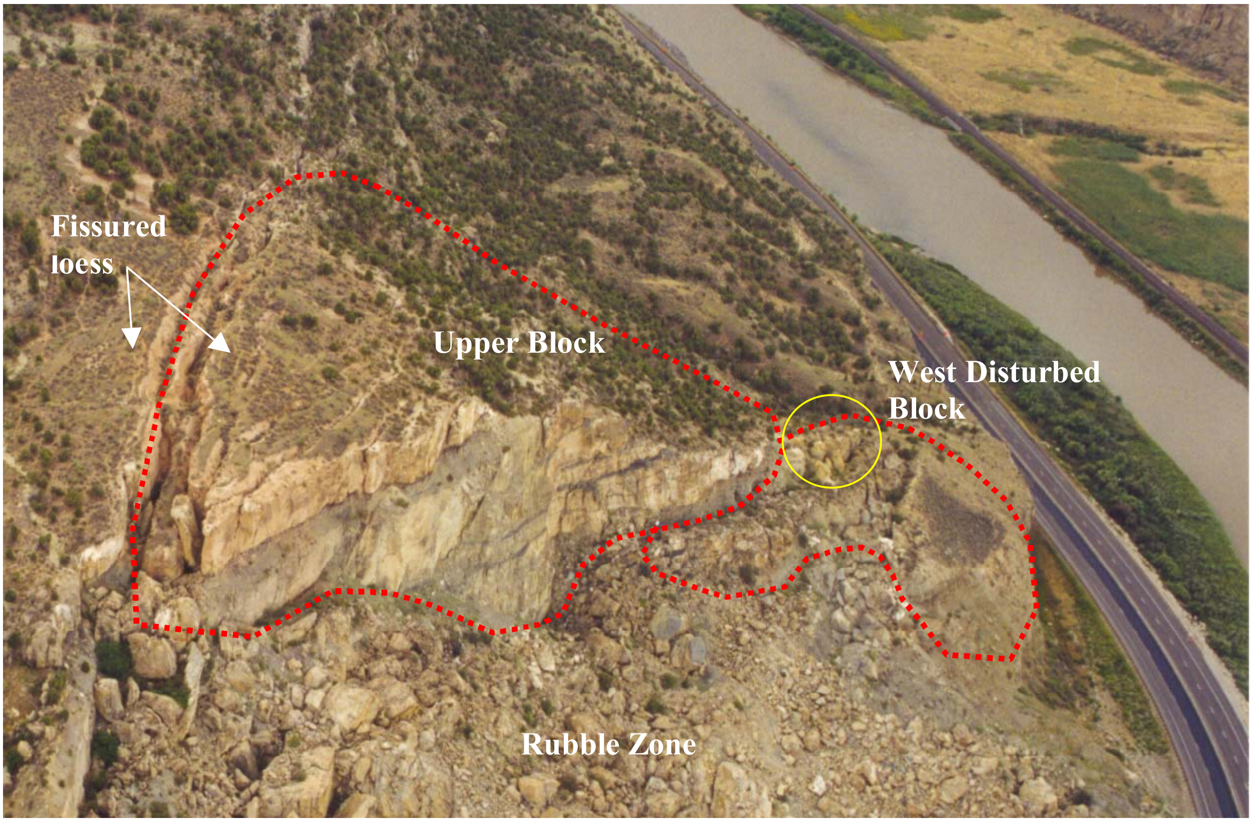


Figure 6. Oblique aerial photo of the Upper Block and West Disturbed Block. Note main graben and the fissures in loess that mantle top of Upper Block. Note also the proximity to Interstate 70, the Colorado River, and the railroad tracks below. Yellow circle is location of mid-Pleistocene (Bull Lake) gravel terrace remnant. Photo taken in 1999 by J. White. View is to the west.

between the Upper Block and the Rubble Zone and is oriented parallel to the major east-west trending joint set.

The Rubble Zone

The Rubble Zone is the main body of the landslide. Lateral and vertical movements at the toe of the rubble have repeatedly damaged the roadway. The Rubble Zone is derived from major block-glide failures, shearing and breaking up of the valley wall, toppling rockfall, and translational movements of the rubble material down from the headwall fissure and the active cliff face of the Upper Block. Chaotic mixing of Mesaverde sandstone, siltstone, and shale during down-slope movements has created a rubbly area of random angular blocks and boulders with a basal clayey matrix layer that is shearing along its contact with underlying weathered claystone. The Rubble Zone can be further subdivided based on landslide mechanisms: an upper translational area, and a lower rotational area.

The upper translational area is characterized by very large blocks, some the size of small houses. This area contains the rubble remains of a much larger rockmass failure, of which the Upper Block Zone is a remnant. Relicts of the main fissure and graben can still be seen at the top along the headwall and are shown on the geologic map plate. The active cliff face of the Upper Block feeds additional rubble into this area. Boring data shows that the mode of movement is

translational, along a buried bedrock slope.
The lower rotational area of the Rubble Zone includes recent rotational slide movements. Large rotational failures have occurred in two directions into the river and caused major deflation of the landslide topography sometime early in the 20th century. Figure 7 is a 1910 photo of the landslide that reveals recent landslide rubble extending into the Colorado River. Major re-activation occurred in 1958 after construction of a modern two-lane highway removed a portion of the landslide toe. In the center of the Rubble Zone, a large hump of material, also referred to as the Nose Area, topographically separates a smaller eastern rotational slide from the main rotational slide to the west. The crest of the Nose Area probably represents the original elevation and form of the older landslide surface, likely reflected in the 1910 photo. The smaller eastern rotational area is similar to the main rotational area that is discussed below.

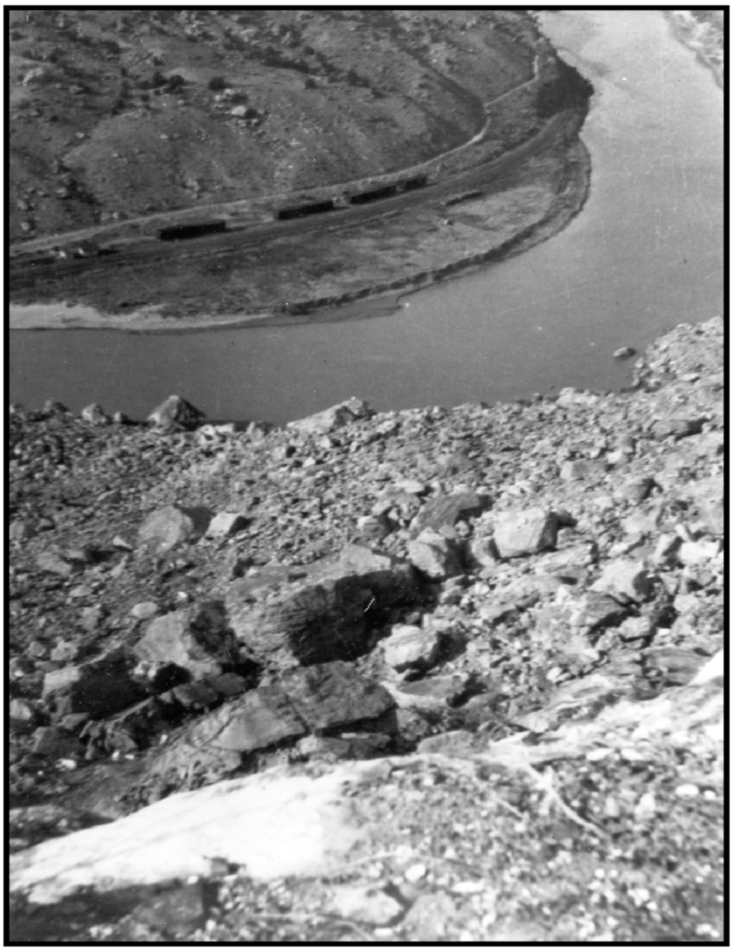


Figure 7. Landslide photo taken in 1910. View is to the northeast looking up the Colorado River with the flow direction from right to left. Note the very recent, fresh, landslide toe in the Colorado River and lack of any reworking of the debris by the river. Also note the light-colored flooding scar on the opposite riverbank. This photo was taken previous to any roadway construction on the south canyon side through the landslide toe. Copyrighted photo is courtesy of the Julia Harris Collection at the Museum of Western Colorado.

The main rotational slide area of the Rubble Zone (Figure 5) was the site of the major movements in 1998. The geologic map plate shows the concentric, listric scarps that indicate substantial recent movements and pronounced deflation of the landslide rubble mass. Boring data indicate that the upper concentric scarps formed in the translational zone as shallow debris slumps moved into a depleted zone of the main rotational area after the main landslide movement left the area deflated. Weathered, disturbed, dark-gray claystone can be seen on the listric surfaces of some of these lower scarps, which is assumed to be portions of the thick weak claystone (shown in the cross sections) that is being incorporated into the rotational landslide mass. One scarp that extends to the cliff line can be seen cutting across much older scarps of the upper rubble area. Where this scarp extends to the active cliff of the Upper Block, soil smears on the lower cliff face indicate that the rubble surface has moved down, relative to the cliff. Cross section A in Figure 3 reveals that the change from translational to rotational slide movement occurred at a buried formational knickpoint, a sandstone cliff that becomes visible west of the landslide. Below the buried cliff, the landslide material thickens and the landslide toe has buried the Colorado river alluvium.

The West Disturbed Block

The West Disturbed Block is located west of the Rubble Zone and directly below (north of) the Upper Block. The West Disturbed Block is a transitional feature in the landslide that has differentially sheared and separated from the Upper Block in a scissors-like fashion. Even though slumping, shearing, tilting, and fissuring have disturbed the rock strata, it has not been completely reduced to rubble and can be easily identified. This feature is identified as the West Block on Figures 5 and 8, and is shown on the right side of Figure 6.

The disturbance of rock strata in the West Disturbed Block appears to be from gliding and shearing within the same thick underlying claystone that is responsible for the block sliding of the Upper Block and the translational movements of the Rubble Zone. Prominent pressure ridges parallel the major shears and scarps in the disturbed block. The original, colluvium-covered shale slope in the disturbed block has been oversteepened, fissured, and disturbed by lateral movements, and small sloughing failures have occurred over the sandstone cliff to the roadway level below. These disturbed shale exposures contain salt deposits that indicate seasonal or temporary seeps in this thick problematic shale. The geologic map plate shows the relationship of the scarps and fissures in the West Disturbed Block as they transition to intermediate arcuate rotational scarps of the Rubble Zone. Deflation of the Rubble Zone has left the adjacent West Disturbed Block at an elevation some 40 ft (12 m) higher than the Rubble Zone surface.

The top of the West Disturbed Block includes remnants of an alluvial gravel terrace. The gravel, also fissured, predates landslide movement and lies 240 ft (73 m) above the present Colorado River elevation. This terrace is likely Bull Lake age (approximately 160,000 BP) based on its elevation above the current river level and its reddish-brown hue (Yeend, 1969). The terrace gravel provides a significant constraint on the age of the DeBeque Canyon landslide complex, as will be discussed later.

PHOTOGRAMMETRIC ANALYSES AND INSTRUMENT MONITORING

Since 1998, the DeBeque landslide has been instrumented with inclinometers, peizometers, vibrating-wire extensometers, tiltmeters, precipitation gages, rockfall warning fences, prism survey stations, and GPS survey stations. Figures 8 and 9 show the locations of the various

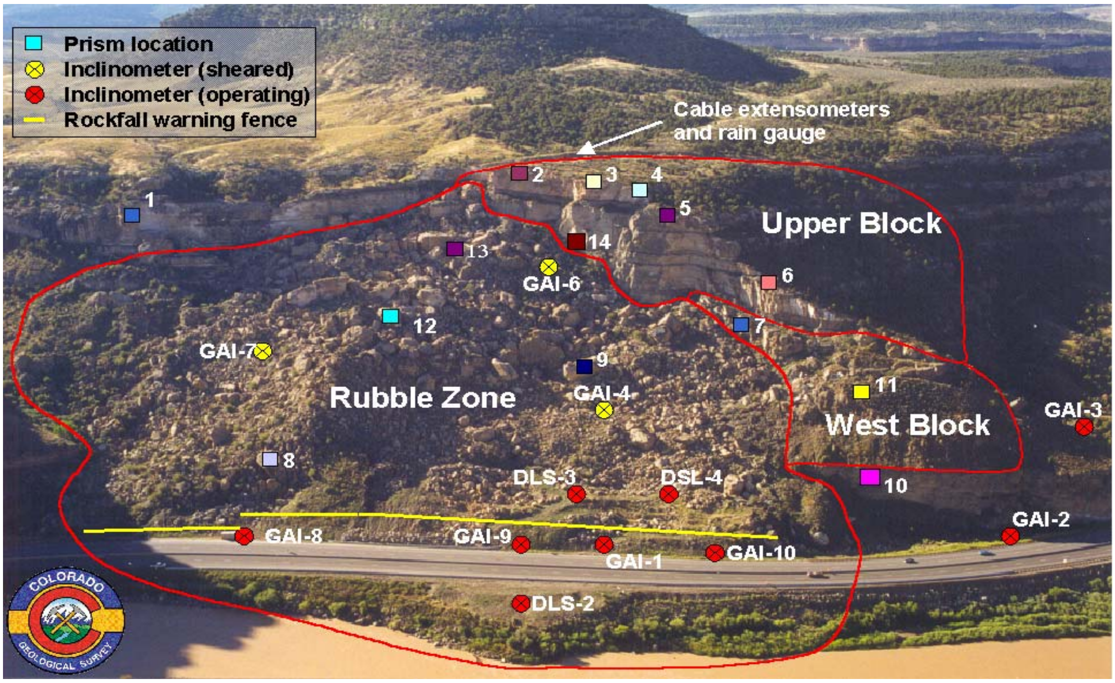


Figure 8. Oblique aerial photo of landslide showing location of various monitoring tools. Photo taken in 1999 by J. White.

instruments and stations on the landslide. Prism station locations are also shown on the geologic map plate. Many of these monitoring instruments are automated, and two data logger stations have been constructed at the site that allow cellular phone communication. Project monitoring for the last four years has shown continued movements of the Upper Block, the upper reaches of the Rubble Zone, and the West Disturbed Block. Inclinometers have now sheared, except for those near the roadway. Figures 10 and 11 present vector graphs of the prism and GPS surveys that have been completed since 1999. They show conclusively that the Upper Block is moving to the north, towards the highway and Colorado River. The data collected also indicate that movement rates accelerate during the late fall and winter, then slow during the spring and summer. These movement variations are likely related to moisture surplus and shadow effects of the north-facing slopes when days are shorter and the sun angle is lower.

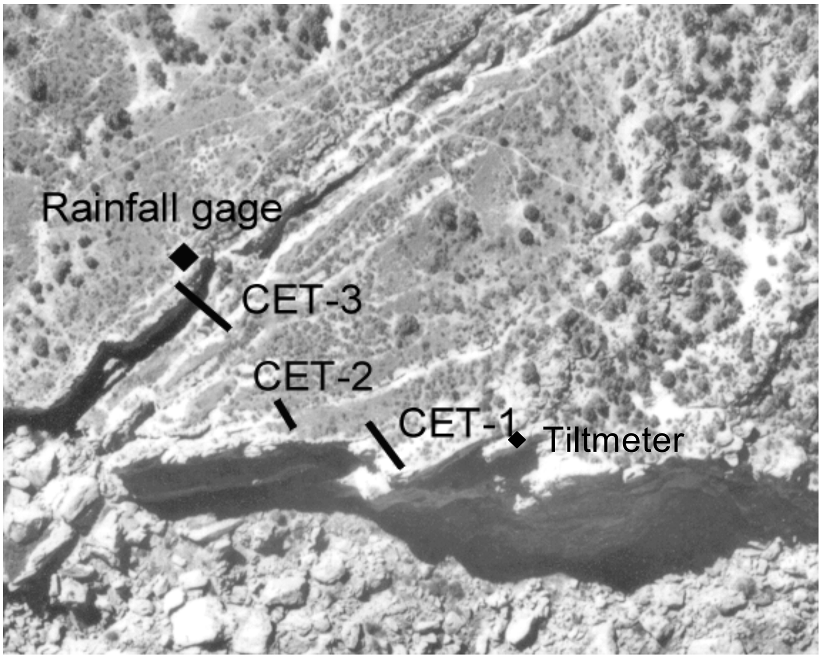


Figure 9. Close-up aerial photo of main graben and Upper Block with locations of cable extensometers (CET), tiltmeter, and base station that includes a rainfall gage.

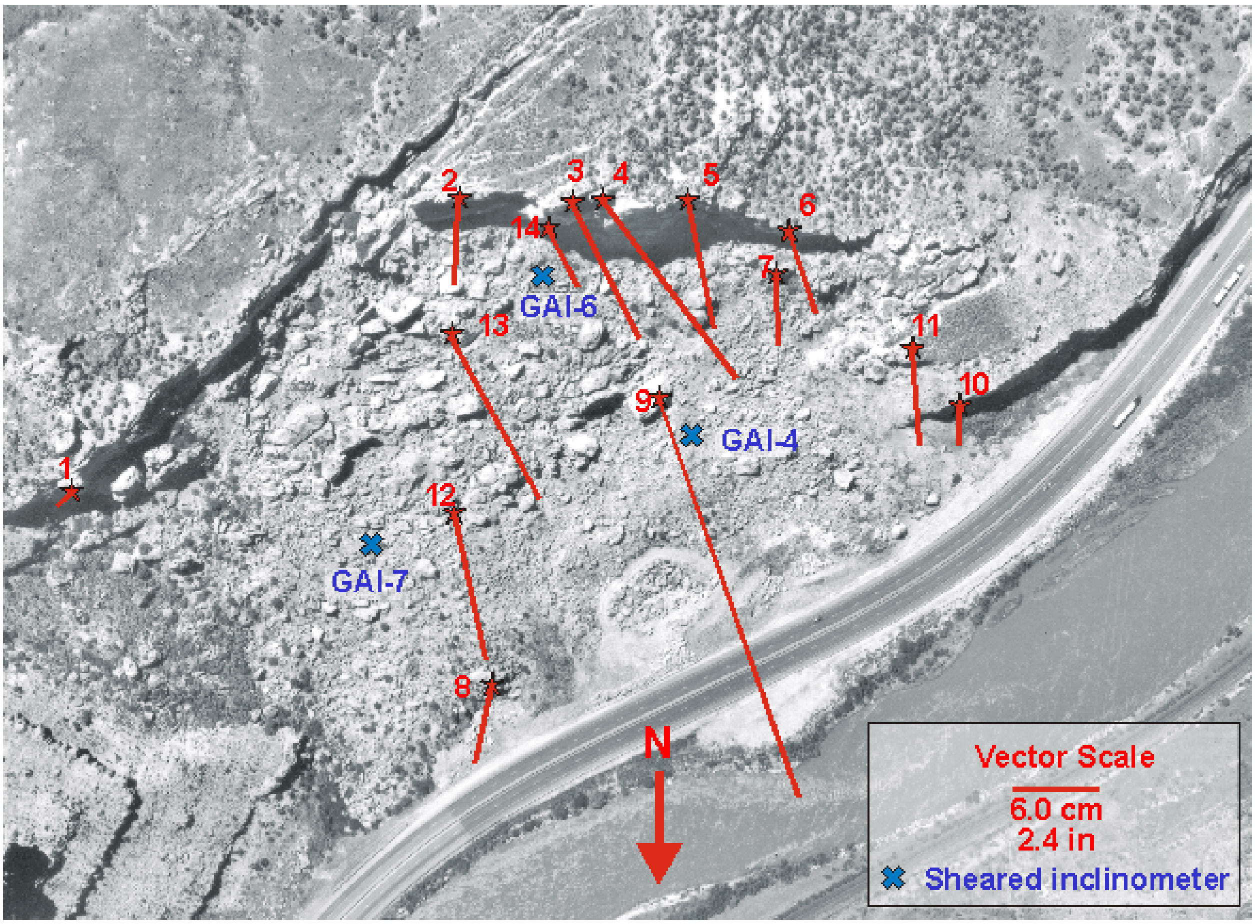


Figure 10. Movement vectors of prisms monitored from August 1999 through August 2002. Total movement vector for Prism #14 is smaller by comparison with the others on the cliff because it was installed 9 months later. The authors believe the Prism #10 vector is misleading. The vector is a mathematical calculation of initial and last-read measurements while the random range of multi-year measurements seems to show that the lower cliff is not moving).

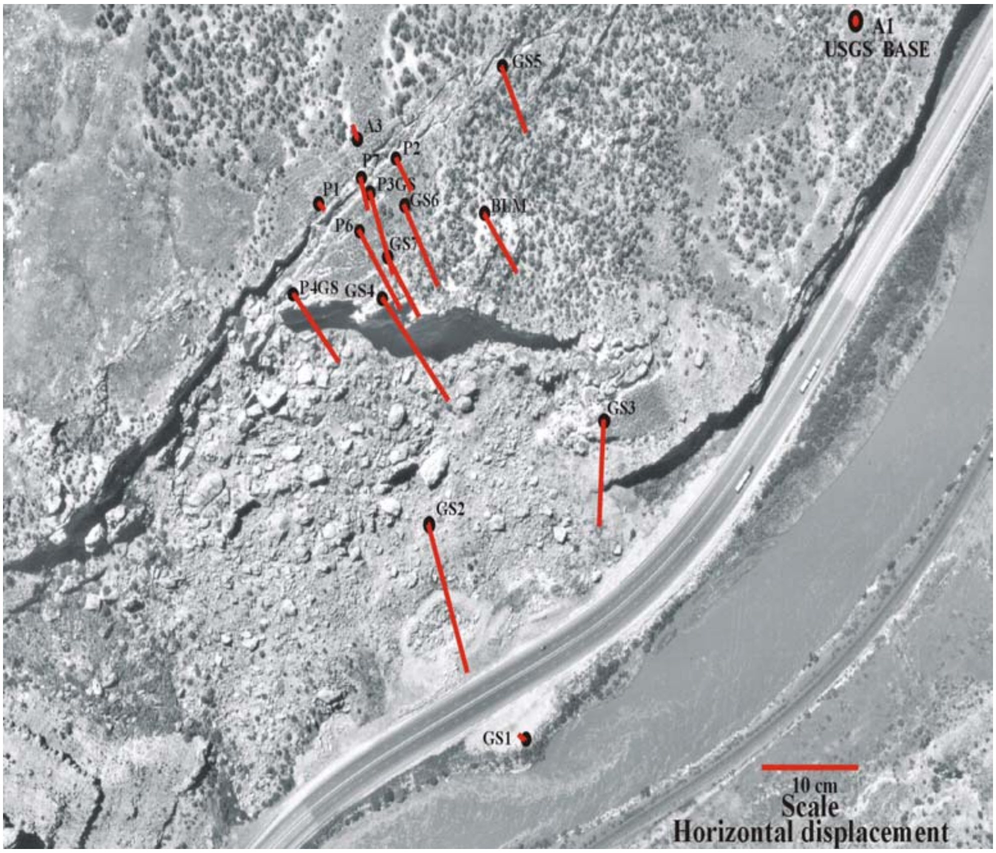


Figure 11. Movement vectors of GPS survey points measured from 1998 to 2002. Stations P1, A3, and GS1 vectors are within the error range (+/-1 cm) of the instrument.

Landslide movement history and implications for the future

One of the reasons for monitoring the landslide was to determine whether current movement rates are wholly in response to short-term re-adjustment of the landslide mass after the major movements that occurred in April 1998, or whether they indicate some form of steady-state movement that results in periodic failure that could be anticipated. Except for periodic aerial photography, no long-term monitoring data exist prior to 1998. Talbott (1969) conducted triangulation surveys of points located on the Upper Block, but reported that they had proven inconclusive for determining movement. He did not include any of the original survey data in his Thesis. Mr. Talbott is now deceased, and the original surveying data could not be located. Talbott (1969) conducted the only comprehensive study done on the landslide prior to this study,

and no long-term monitoring of ground movements have been conducted in the interim. Although previous movement events are known and relative dating evidence suggests other ground movements, there are no quantitative measurements to indicate whether the landslide moved only during significant events such as in 1958 and 1998, or that creep-type movements occur continuously. There is also no data to indicate whether or not creep accelerates prior to, or after major reactivations. Obviously therefore, no definitive data exists that might constrain acceleration rates of possible creep before or after major reactivations of the landslide. Analysis of several sets of historical aerial photography, however, has provided some insights into recent pre-1998 activity of the landslide.

During the research phase of this project, Golder Associates (2000) acquired many sets of stereo aerial photography of the landslide site, dating from 1937, 1950, 1958, 1966, 1967, 1979, 1981, 1985, 1993, and 1994. A final set of aerial photographs from 1999 was commissioned by the CGS for this study. A cursory review of these photographs indicates that the landslide looks basically the same as it did in the 1937 photography. However, careful examination and systematic stereographic analysis of the photography revealed significant changes to the landslide over time. Aerial photos of the landslide from these earlier years are shown in Figure 12.

1950: Note highway construction of Highway 6 and 24 through toe of landslide.

1958: Photo taken shortly after 1958 re-activation of landslide. Photo shows re-alignment of Highway 6 and 24 and grading of landslide slope.

1967: Photo shows old Highway 6 and 24 passing through toe of landslide.

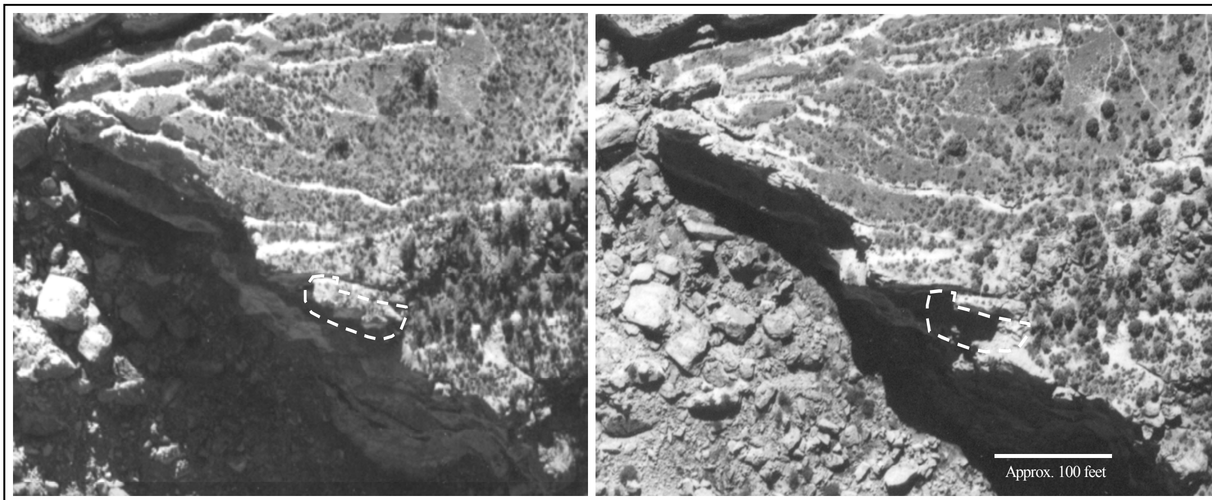
1999: Photo shows repair and buttressing of Interstate 70 following the 1998 re-activation of the landslide. Note darker asphalt where road repair occurred.

Figure 12. Aerial photography of the DeBeque Canyon landslide taken in various years. Early photograph images are courtesy of the Colorado Aerial Photography Service.

The landscape changes identified in the aerial photography were:

1. Changes in the location of the highway and new cuts into the landslide toe from new highway construction and reconstruction after landslide events.
2. Changes in the surface of the Rubble Zone, particularly in the main rotational zone, that indicated a general deflation of the ground surface.
3. A large rockfall from the active cliff face that separates the Upper Block from the Rubble Zone.
4. General downward movements of identifiable boulders within the Rubble Zone.

The large rockfall noted in item 3, one of the more dramatic changes visible in the photos, can be seen in Figure 13, which shows the location and morphology of the cliff face in 1967 and 1999. A large sandstone block, approximately 75 x 25 x 45 ft (23 x 8 x 14 m), detached from the face and fell into the Rubble Zone sometime during this time interval (1967-1999). Further examination of higher altitude aerial photography more tightly constrains this rockfall to the time interval between 1981 and 1985. This rockfall may have gone unnoticed because there is no CDOT maintenance record of the 3,200 cy (2,447 m^3) rockfall event. Apparently the roughness of the 700-ft (213 m) slope below the cliff prevented any rocks from reaching the highway.



1967 aerial photograph of Active Cliff Face. Note area inside dashed white line and compare with adjacent 1999 photo.

1999 aerial photograph of Active Cliff Face. Note portion of cliff edge missing compared to the 1967 photograph. Sliver of rock remaining shows continued creep and is instrumented by a tiltmeter and also monitored by prism #4.

Figure 13. Comparison of Active Cliff Face in 1667 and 1999 aerial photographs. Note missing sandstone block in 1999 photo.

Because the photographic records documented significant movements of the landslide over the last 62 years, a plan was developed to use a more rigorous, quantitative method of air photo analysis to derive more precise measurement of historic landslide movements over time.

The semi-arid to arid climate and sparse vegetation at the site made it possible to identify several common points on the various sets of aerial photography that could also be located on the ground. These were surveyed and then used as ground control for precise photogrammetric analyses of observable changes in the aerial photography. Photogrammetrically-derived X,Y,Z coordinate data from the pre-1999 aerial photography, in the same projection, were superimposed on the existing (1999) ground terrain using GIS software. This method results in a relatively precise measurement of landslide movements over time, a 'snap shot' of sorts of the conditions at the times when the earlier sets of aerial photography were flown. The CGS had previously plotted the geologic map photogrammetrically at the USGS Laboratory for Geologic Photogrammetry and Digital Mapping at the Denver Federal Center. The resident cartographer/geologic photogrammetrist set up the analytical photogrammetric models for aerial photography dating from 1950, 1958, 1967, and 1999 and provided raw X,Y,Z data points along referenced profiles, three of which corresponded to mapped cross section lines B, C, and D shown on the geologic map plate. Photogrammetric profile A was used as a control and does not correspond with profile A shown on the geologic map plate.

The CGS GIS Division received this data and incorporated it into a GIS project that had been prepared for the DeBeque landslide project. Using ArcInfo, ArcView and the ArcView 3D Analyst extension, a digital elevation model (DEM) was rendered from the 1999 photogrammetric data. The 1999 aerial photography was geo-referenced and draped over the DEM. The text files that contained the profile X,Y,Z coordinate data points were converted to cross section lines and, using GIS software, projected in the same geo-referenced three-dimensional space (Figure 14). Figures 15, 16, 17, and 18 show the various landslide movements that have occurred, based on this geo-referenced data set that superposes several years of aerial photography on the 1999 DEM.

In addition to constructing surface profiles along the selected cross sections for the variously dated aerial photographs, seven boulders identifiable in the photos were selected for tracking in the four photogrammetric models to determine rates of lateral movements in the Rubble Zone. The 1950 data proved to not be reliable for this measurement, and only movement data from the 1958-1967 and 1967-1999 time periods could be determined. From 1958 to 1967, a period of time without a major landslide reactivation, the boulders moved from 1.4 to 4.0 ft (0.4 to 1.2 m), and from 1967 to 1999 they moved from 3.5 to 60.6 ft (1.1 to 18.5 m) downslope as projected to a horizontal plane. Figure 19 shows the boulders and movement rates derived from the 1958, 1967, and 1999 photogrammetry.

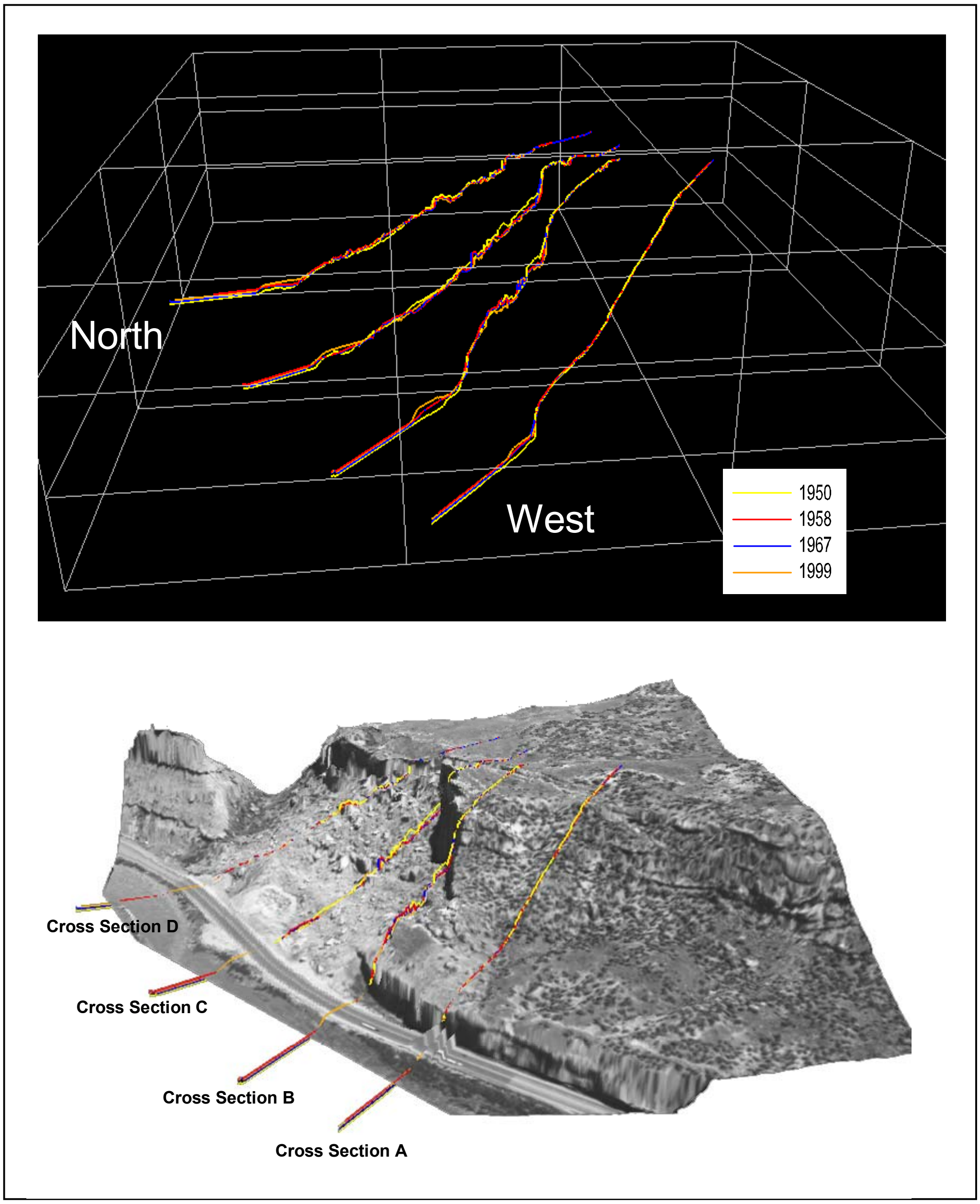


Figure 14. Projections of photogrammetric cross sections in 3D space and on DEM with draped 1999 aerial photograph. Cross sections B, C, and D correspond to same-lettered cross sections shown on geologic map. Cross section A is a control profile and does not correspond with profile A on the geologic map.

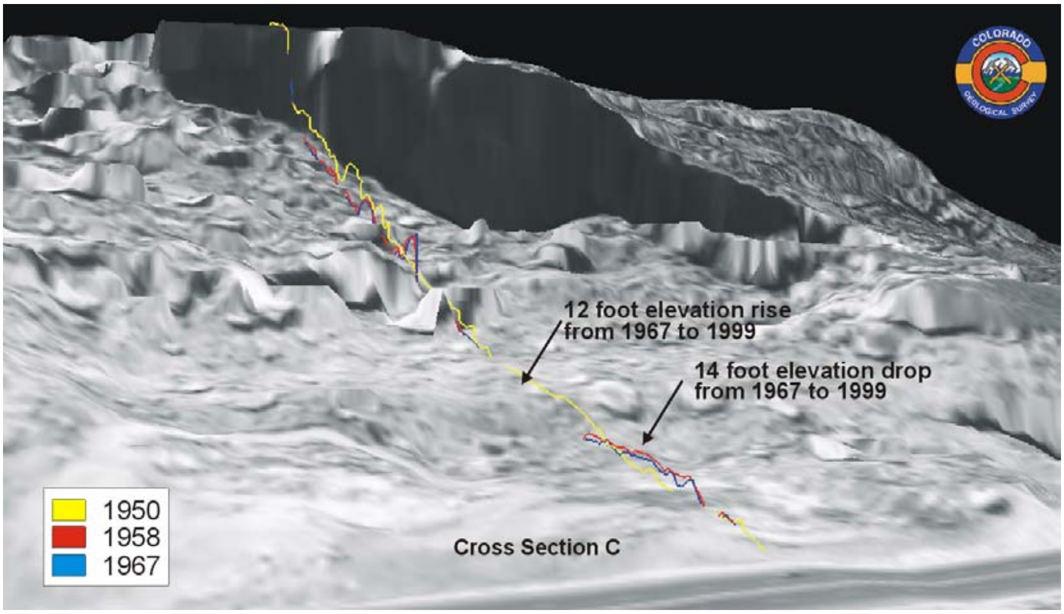


Figure 15. Oblique 3D view of DEM with draped 1999 aerial photography at the main rotational area. Distortion is caused by drape smear effect at steep slope areas. Colored lines are Cross Section C. Where lines disappear or are not shown, the 1999 ground elevation is higher than the elevation from previous years. Note marked deflation of Rubble Zone as compared to 1950 (yellow) line and downward migration of large rock block. Also note that the area midway in main rotational area is inflated compared to 1958 and 1967 data, but has deflated below along a well-defined scarp, where major rotational failure of 1998 lowered this part of the landslide.

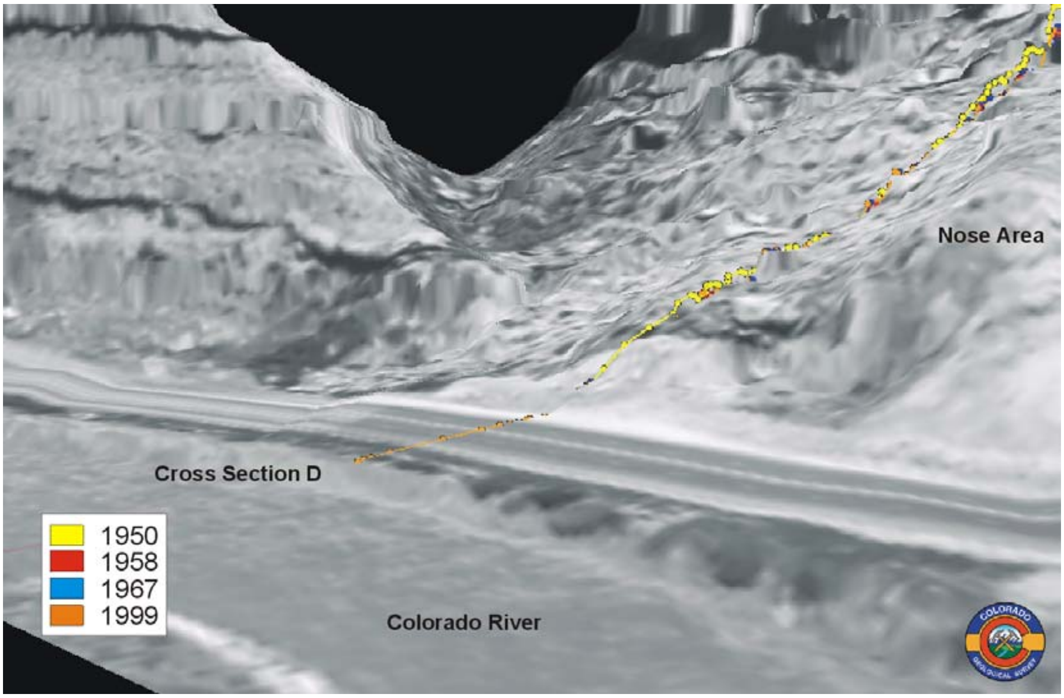


Figure 16. Cross Section D at east rotational area. Model reveals that majority of the landslide surface was higher in 1950 than in 1999. Plots of 1958 and 1967 elevations on Cross Section D are predominantly below current 1999 elevations and are not visible in this rendering. Creep continues in the east rotational area and inflation of the 1999 surface, compared to 1958 and 1967 surfaces, would seem to indicate that the area is being loaded by translational movements of rubble from above.

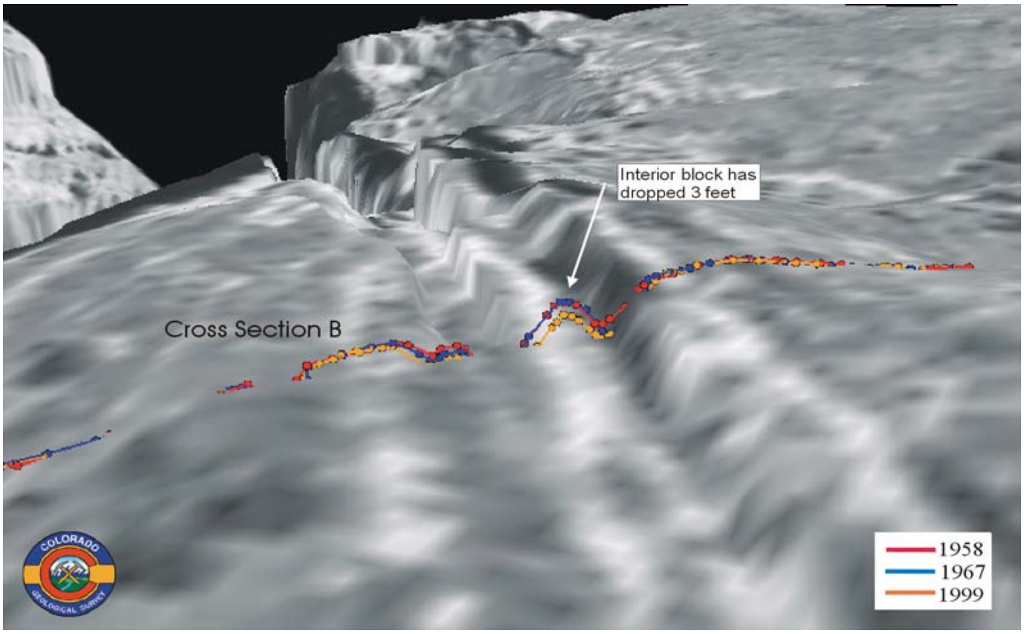


Figure 17. Main Graben Fissure above Upper Block. Photo drape is noticeably skewed by DEM conversion of poor quality photogrammetric contour lines at fissures that are more than 50 feet (15 m) deep along Cross Section B. Points of 1999 cross-section line (tan-colored) are a more accurate reflection of actual ground surfaces. Note 3-ft deflection downward of interior block of graben from 1967 to 1999, and little movement from 1958 to 1967. Also note left lateral movements of the Upper Block.

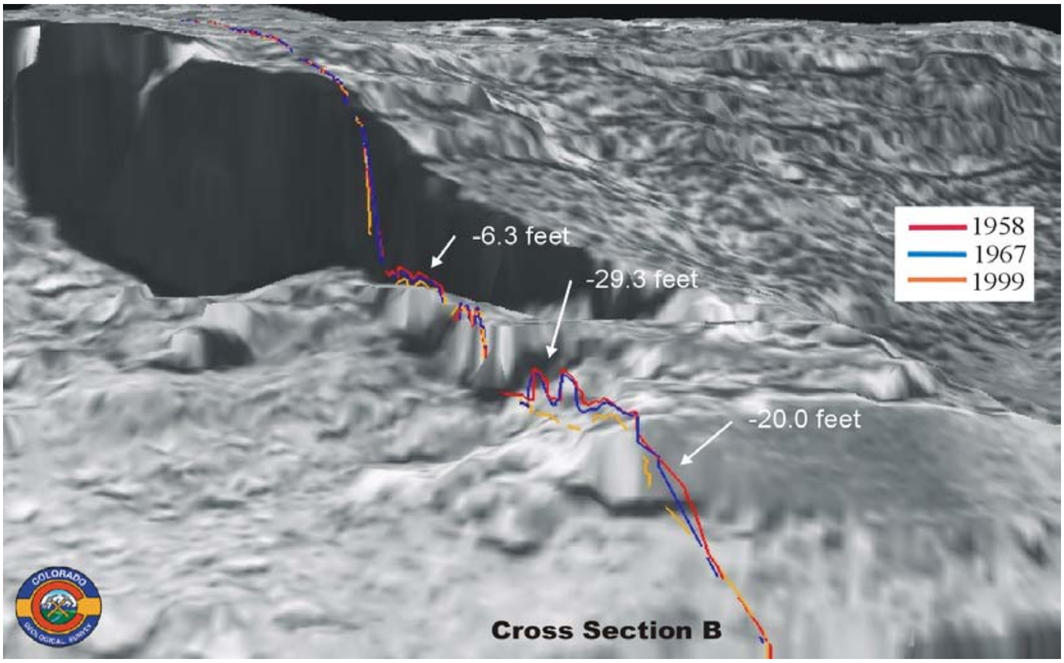


Figure 18. Cross Section B through the West Disturbed Block. White arrows and footage offsets are from elevation differences from 1958 to 1999 along the cross section line. Note blocks missing in center of image, which is reflected in the 1958 and 1967 plots. These large boulders have subsequently toppled into the Rubble Zone, and thus are not reflected in the 1999 plot data.

Boulder	1958-1967 Movement (ft)	1958-1967 Average (ft/yr)	1967-1999 Movement (ft)	1967-1999 Average (ft/yr)
1	3.83	0.43	60.6	1.89
2	2.91	0.33	6.3	0.19
3	1.40	0.16	8.8	0.28
4	ND		5.6	0.18
5	2.98	0.33	5.5	0.18
6	4.0	0.44	4.2	0.13
7	1.83	0.20	3.5	0.11

Figure 19. Photo and table of boulder movements measured photogrammetrically as horizontal displacement down the slope.

Discussion and Interpretation of landslide movement data

The data presented indicates that movement of the DeBeque Canyon landslide complex is a continuous and dynamic process. Many changes in elevation and morphology were identified in aerial photographs of the main rotational area of the Rubble Zone and the West Disturbed Block. Smaller, subtle changes were seen in the east rotational area. It is clear that the downward creep of material in the translational area of the Rubble Zone inflates the landslide mass at the head of the rotational area of the Rubble Zone. This accumulation increases the driving forces of the landslide and when triggered, most likely from a groundwater threshold level in the landslide deposit, reactivates shearing along circular slip planes. Such rotational movement heaves the road up in the toe area.

Comparison of the 1950 and 1958 data showed tens of feet of deflation within the interior of the

Rubble Zone and also revealed downslope movements of boulders. This is consistent with the known re-activation that occurred in 1958, shortly before the 1958 aerial photography was flown. The data from 1958 and 1967 reveal a less-active landslide, but some downward movement of boulders and general inflation continued. This is consistent with the interpretation that translational movement of debris from higher in the Rubble Zone to lower in the Rubble Zone is a continuous process that inflates the lower part of the Rubble Zone (rotational area). Because of the quality of the available aerial photography, quantitative data from the middle 1990s was unavailable to measure the landslide inflation at the top of the rotational area prior to the re-activation of 1998. Consequently, the inflation threshold at which the 1998 failure occurred remains unknown. The 1999 photography, after the 1998 landslide re-activation, shows areas within the main rotational area at lower elevations than they are shown in the 1967 photography. These deflated areas in the 1999 photography correspond closely to an intermediate scarp that defines the major rotational failure that occurred in 1998. Above this scarp, at the base of the translational area, the 1999 surface elevation is higher than in 1967. Farther above this scarp, the 1999 landslide surface elevation again becomes lower than in earlier years.

Movement of the Upper Block was measured by selecting cliff edge points on the photography sets. Based on analysis of post-1967 aerial photography, the Upper Block has been creeping northward at an average rate of 1.1 in (2.8 cm) per year since 1967. That is substantially less than today's rate that was monitored along the cliff edge in 1999-2002 by the current instrumentation (i.e., Prism #3 measured 1.8 in/yr (4.5 cm/yr)). GPS surveys of stations in the interior of the Upper Block show ranges from 1.1 to 1.6 in/yr (2.9 to 4.1 cm/yr). Both tools have an error range of about 1 cm. Another source for estimating creep of the Upper Block is a barbed-wire fence gate opening that straddles part of the main fissure (site is shown on geologic map plate). The gate opening has widened by about 20 in (50 cm) since the gate was installed (Figure 20). If an age range of 40 to 60 years is assumed for the fence and gate, an average movement from 0.5 to 0.33 in (1.3 to 0.85 cm) per year can be inferred. This rate is lower than the rates mentioned above, but the fence site is in the western portion of the Upper Block where movements are not as pronounced, and also the gate does not completely straddle the main fissure zone. There is not yet enough data to definitively characterize current and past movements of the Upper Block and also determine whether or not multiple slip planes may exist in different shale layers. An important question, however, is whether or not creep of the Upper Block has accelerated since the 1998 re-activation. Though the data is limited, it suggests that lateral creep of the Upper Block, and perhaps downward movements of rubble in the translational area of the Rubble Zone, are accelerating in response to the unloading caused by the 1998 re-activation of the rotational area of the Rubble Zone.

Prism and GPS survey data suggest that the Upper Block may be moving differentially, in that movement in the central and eastern end of the block is somewhat more rapid than on the western end. A toppling component along the active cliff face may account for some of this movement. This is consistent with physical observations that indicate the Upper Block movement is splayed, with more abundant and disperse fissuring in the eastern part of the block and with a pivot point in the far southwestern portion of the landslide where the main fissure disappears. This may indicate that the sandstone cliff below the West Disturbed Block and the undisturbed slope further west are critical features in the landslide that buttress the Upper and

Figure 20. Photo of barbed-wire gate showing offset. Subjects right hand is holding wire loop that fence post originally was placed into to close the gate. Note fissures below the wire-fence gate. Photo taken in 1999 by J. White.

West Disturbed Blocks and control their ability to move catastrophically. The survey data from prism (#10) installed on the sandstone cliff below the West Disturbed Block in September 1999, indicates that either no movement has occurred there, or the movement is so small it falls within the inherent error range of the survey tool. The inclinometer casing in boring GAI-3, shown in Figure 8, does not show any differential movement between the ground surface and a depth of 150 ft (46 m).

The current Interstate 70 roadway elevation approximates the elevation of the heaved landslide surface after the 1998 reactivation, and there are no visual indications of post-1998 movement in the roadbed or asphalt surface. The nested inclinometers GAI-8, 9, and 10, the manual inclinometer GAI-1, and a GPS survey point (GS1 in Figure 11) have also not indicated any movement on the toe of the landslide. This suggests that the rotational part of the landslide is currently in equilibrium. During repairs to the highway following the 1998 event, fill was added to the toe of the rotational slide, providing a buttressing effect. Loading continually occurs as the translational movements in the upper part of the Rubble Zone move material downslope into the rotational area of the Rubble Zone, as indicated by the sheared inclinometers. For the Rubble Zone, these relations suggest a simple failure model of loading followed by rotational failure, a process that appears to be continually repeated. In this model, the translational part of the Rubble Zone acts as a conveyor of material to the rotational area (Figure 21). During the 1998 reactivation the Rubble Zone also pulled away from the base of the Upper Block and confining pressure and lateral resistance against the underlying weak shale was lost. Data and observations previously presented suggest that the Upper Block may now be in a state of accelerated creep. If this apparent acceleration in creep is related to loss of confining pressure against the underlying

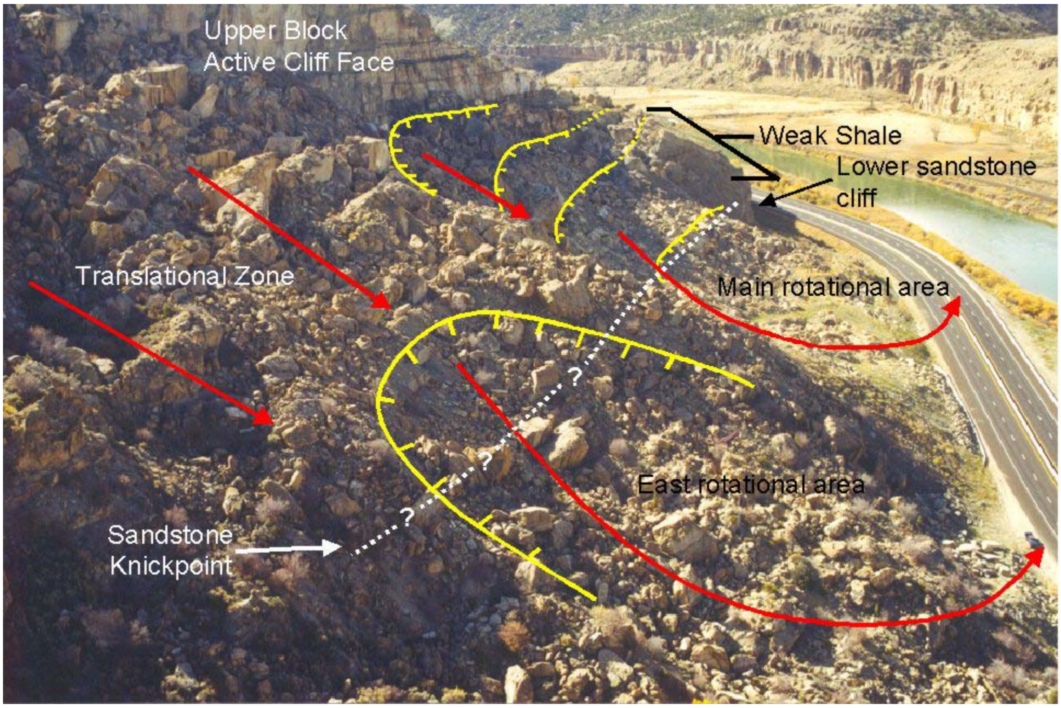


Figure 21. Oblique photo showing mechanism of landslide movements. Photo taken in 1999 by J. White. View is to the west.

weak shale, then the shale may become further destabilized and increase the likelihood that large blocks of sandstone could calve off the Upper Block and further load the Rubble Zone.

Most of the societal significance of the DeBeque Canyon landslide is derived from its impingement on Interstate 70, the Colorado River, and the railroad on the opposite side of the canyon. These features are known to have been impacted in the past by the landslide and they will probably be impacted again at times in the future. Continued monitoring of the landslide and Upper Block movement should lead to a better understanding of the landslide and improved mitigation of the hazard by forewarning when landslide activity changes, either subtly or radically.

RELATIVE AGE OF THE LANDSLIDE

CDOT and FHWA engineers requested information about the age of the landslide complex and whether the large fissures in the Upper Block (graben) were a result of highway construction in the mid to late 1900s**.** The historic record, photogrammetric analysis, and current monitoring reveal that movement continues in all the major parts of the landslide, which include the Rubble Zone, the West Disturbed Block, and the Upper Block. Further support for continued movement comes from varied physical evidence of ground disturbance, such as tilted trees and shrubs, the offset barbed-wire gate, and fresh soil pressure-ridge scarps that indicate recent movements and continued creep in the Upper and West Block Zones. However, this evidence does not indicate the age of the landslide. Relative ages of several features suggest that all of these landslide features post-date the Bull Lake glaciation (about 160,000 BP) and further suggests that most of the landslide is probably younger than mid- to early-Holocene (5,000 to 11,500 BP). The major parts of the landslide complex may not have all begun to form at the same time as advancing failures and ground movements progressed though the landslide mass, but the major landslide forms existed well before railroads or highways were constructed beneath them.

If it is assumed that the fissured terrace gravel 240 ft (73 m) above the current river level (figure 6) is Bull Lake (about 160,000 BP) in age (Yeend, 1969), then further age-constraining calculations of canyon downcutting can narrow the range of possible ages of the landslide. The problematic thick shale (shown as the weak shale in Figures 3 and 4 cross-sections and Unit B on the map plate) was determined (Golder Associates, 2000) to be the weak link and controlling stratum in slide mechanisms for the Rubble Zone movements, gross disturbances of the West Disturbed Block, and likely creep and fissuring of the Upper Block. The terrace elevation is above this shale so downcutting through it and exposing it to destabilization occurred post-Bull Lake (160,000 BP). Recent quadrangle geologic mapping by the USGS and CGS has Pinedale-aged (about 12,000-35,000 BP) terraces placed from 19 to 50 ft (5.8 to 15.2 m) above the present elevation of the Colorado River (Kirkham et al. 1997, Scott and Shroba, 1997, Shroba and Scott, 1997). No remnants of Pinedale terrace treads were observed in the vicinity of the landslide in DeBeque Canyon, but their elevations would correspond with river cutting into the lower massive sandstone below the problematic shale. From the historic aerial photography (Figure 12), one can observe that the river channel has been displaced by landslide debris, and that the outside and more-erosive curve of the river channel cut into the valleyside prior to major landslide movement. This late Pleistocene erosion into the river bend, at Pinedale-age elevations above existing Colorado River mentioned above, would approximate the elevation of the lower sandstone cliff. This erosion may have oversteepened and destabilized the weak shale slope above the cliff, implying that major activity of the landslide complex post-dates late Pleistocene times.

It is likely that subsequent Holocene destabilization of the oversteepened overlying shales occurred after the late Pleistocene river cut laterally into the sandstone below and the opening of the east end of the main fissures began. The authors are reasonably confident that the bulk of the DeBeque landslide complex in its present form, especially the graben and fissuring seen in the Upper Block, is Holocene in age, late Pleistocene at the earliest. The lines of evidence supporting this conclusion are:

1. By examination of the soil development of the loess mantle, the single layer of loess is estimated to be late Pleistocene (Pinedale). This is based on observation of only a weak Bk horizon and absence of a strong caliche K horizon (Shroba, 1994). Small patches of undisturbed loess of the same age (based on soil development) were also observed in the slope area west of the landslide area near a 4-wheel drive track, below the elevation of the Bull Lake terrace remnant;
2. The main graben fissure cuts through the loess mantle at the head of the landslide complex on the Upper Block Zone, revealing that the loess was deposited antecedent to fissuring; and
3. Lack of loess deposition in the depressions that were created by landslide movements.

Further relative dating of the landslide complex can be made by observation of the fissure surfaces, at both the intact west end at the Upper Block and remnants of the fissure still visible above the Rubble Zone.

1. The south-facing fissure surfaces in the interior of the graben remnants are much more heavily and differentially eroded at the east end above the Rubble Zone, compared to the west end at the Upper Block. Such weathering would take many hundreds, if not thousands of years to occur.
2. There is heavy varnish and discoloration, but lack of appreciable erosion on the headwall cliff face above the Rubble Zone where two channels drain a combined 50 acres (20 ha) from the plateau above. These channels were intercepted by the fissuring and opening of the graben. One would expect significant clefts eroded into the sandstone if these drainage ways poured over the cliff for even a few thousand years. The immature age of the clefts are inconsistent with the more mature drainage channels that flow to them from the plateau above.
3. The rock surfaces of the fissure walls in the graben interior at the Upper Block, near the center of the landslide, are relatively fresh with only modest lichen cover, suggesting an age of only a few hundred years. This conservative estimate is based on the growth rates of Rhizocarpon lichen (Bull and Brandon, 1998) adjusted for semi-arid climates.
4. The freshest cracks and rock surfaces occur near the western end of the landslide where the main fissure begins to tighten and curve downslope. Those surfaces appear to have been opened by the recent major movements in the last century.
5. Further evidence of the youth of fissures at the west end is indicated by exposed, dead tree roots along the sides of fissures. Only very recently fissured ground would expose roots that have not yet rotted away.
6. Carbon-14 dating by the USGS of wood fragments taken from the bottoms of fissures near the west side of the Upper Block, between the cross section B traverse and the offset fence gate, date to 460 BP (+/- 50 BP) (Ellis and Coe, unpubl.). These wood fragments were lying on an unknown thickness of soil in-fill in the fissures, so actual fissure age may be on the order of several hundred years older.

Based on the above observations, the following theory is proposed to illustrate the time-line of this landslide complex. Middle Pleistocene downcutting of the Colorado River left a gravel terrace remnant above the weak shale stratum. The weak shale was not disturbed and no landslide existed. Downcutting of the Colorado River through the weak shale occurred in late Pleistocene. The lower sandstone cliff at the outside margin of the river bend was eroded near the end of the Pleistocene and oversteepened the shale above. This shale (and overlying package of sandstone beds that rim the canyon wall) was then exposed and laterally unconstrained. Lateral movements towards the river began to occur in the weak claystone beds of this shale. A fissure, along a pre-existing (Laramide-aged) shear zone began to open from the unnamed box canyon to the east and propagated westward through the overlying sandstone beds to the rim of the canyon where late Pleistocene loess was also fissured. As the evidence above suggests, the fissuring of this main graben likely began in the Holocene (no more than a few thousand years ago) and the progression was much like a zipper opening from east to west. The westward migration and opening of the fissure intercepted existing drainage channels and allowed the flow of water from the small basins above into the fissure and the underlying weak shale. Lateral movements of the block resulted in additional oversteepening of the mass above the sandstone cliff at the near-current river level.

Recent failures of the canyon rim inside the fissure resulted in the partial disintegration of the landslide block. In the eastern portion of the landslide complex, where maximum movement of the fissure occurred, and the major drainage channels were intercepted, the landslide likely mobilized as several reactivations or movement pulses, and was nearly completely transformed to rubble. Sometime before 1910, this rubble material slid over the lower sandstone cliff and partially blocked and altered the natural bend of the river (Figure 7). Left remaining was the western portion, now referred to as the Upper Block, where the fissure width was smaller and no drainage channels were intercepted to moisten and weaken the claystone below. The Rubble Zone separated from the Upper Block along the trend of the second major joint set (the active cliff face). The Upper Block remained more-or-less intact, with the exception of some extensional deformation. The West Disturbed Block is a transitional feature between rubble and nearly intact rock of the Upper Block.

Subsequent types of landslide activity now differed. The now-buried, lower, sandstone cliff acted as a knickpoint in the landslide rubble and, while translational movements of the rubble still occur above this knickpoint, rotational slip planes developed in the thick landslide deposits below the cliff near the river level. Such accounts are in the historic record that began sometime at the turn of the 20th century when the east and main rotational areas formed, deflating the landslide rubble except at the divide between them, called the Nose Area. The remnants of the original landslide block, the Upper Block and West Disturbed Block, continue to creep towards the river. Active downward translational movement of the Rubble Zone undermines the base of the Upper Block, and causes toppling failures that feed additional material to the Rubble Zone.

MITIGATION

FHWA and CDOT's goal for the engineering study was to develop a mitigation plan for the landslide. Golder Associates, Inc. facilitated a formal decision analysis process as a culmination of a workshop dedicated to this task. This workshop included representatives of the major task participants and CDOT Region 3 engineers. An outcome of this workshop was the formal recommendation to adopt CDOT's Emergency Road Closure Plan if sudden catastrophic failure impacted the highway. Two other near-term mitigation actions were also recommended: (1) diversion of surface water away from the landslide, and (2) continued monitoring.

While difficult to quantify the effectiveness of intercepting storm flows from ephemeral streams that drain into the landslide, common sense dictates that removing moisture from landslides is beneficial. From a cost basis, diversion of water was by far the least expensive technique available for mitigation of landslide activity. Groundwater and overland flows of water into the landslide mass are typical triggers for landslide movement. Natural drainage channels above the landslide complex flow into the main graben fissure within the Rubble Zone, where more and increasingly chaotic landslide movement has obviously occurred compared to the Upper Block.

In 2001, CDOT developed a project design to intercept three different drainage basins that flow into the landslide and divert these flows into a diversion ditch excavated to the unnamed box canyon to the east. This construction project was completed in the summer of 2002 (Figure 22). CDOT continues to fund CGS and provides partial funding to USGS to continue monitoring of the landslide complex.

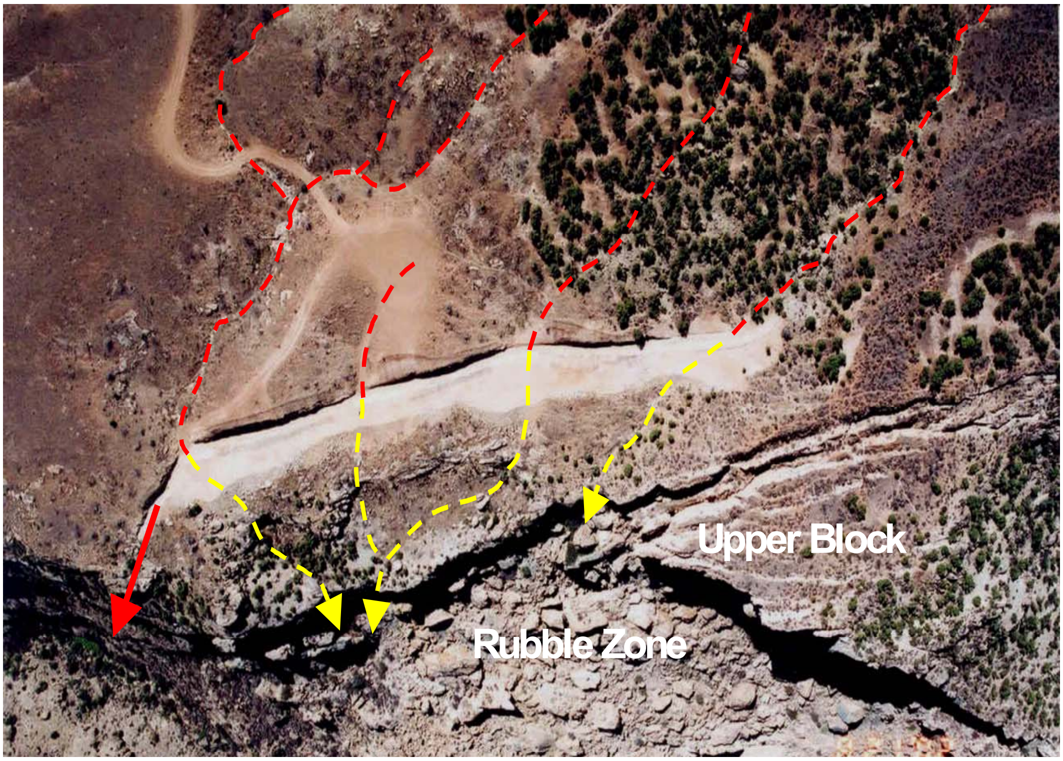


Figure 22. Water diversion structure constructed in Summer and Fall of 2003 to intercept drainage basins above landslide. Dashed red line of existing channel locations become dashed yellow where intercepted. Red arrow shows out-fall of diversion ditch over canyon rim to unnamed box canyon east of the landslide complex. Note that original drainage channels flowed into the Rubble Zone. Aerial photo courtesy of Robert Florez and the CDOT aerial reconnaissance unit.

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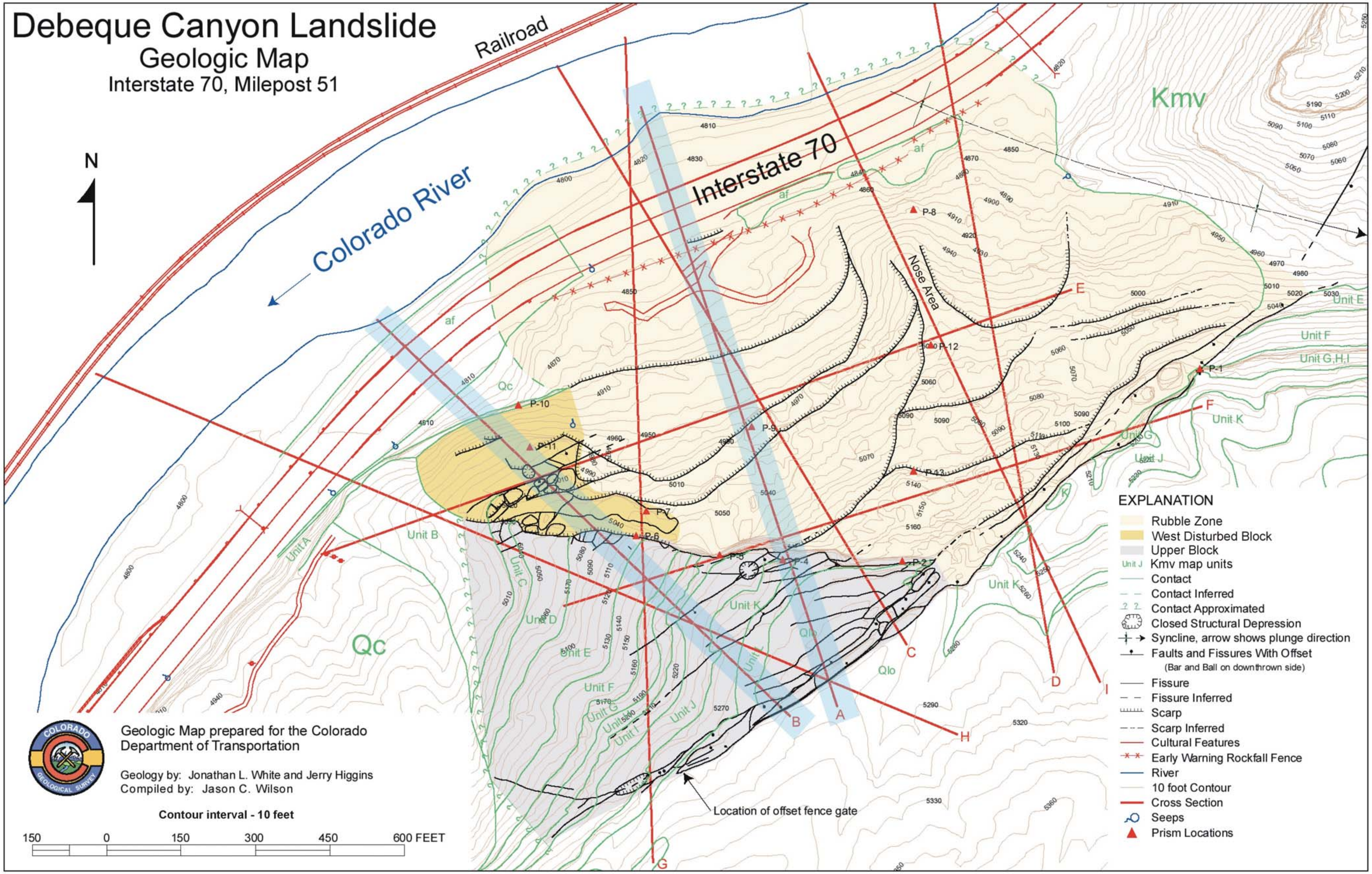
Debeque Canyon Landslide
Geologic Map
Interstate 70, Milepost 51
Railroad
Colorado River
Interstate 70
Kmv
Qc
Qlo
Nose Area
af
Unit A
Unit B
Unit C
Unit D
Unit E
Unit F
Unit G
Unit G,H,I
Unit I
Unit J
Unit K
P-1
P-2
P-4
P-5
P-6
P-7
P-8
P-9
P-10
P-11
P-12
P-13
A
B
C
D
E
F
G
H
I
N
EXPLANATION
Rubble Zone
West Disturbed Block
Upper Block
Kmv map units
Contact
Contact Inferred
Contact Approximated
Closed Structural Depression
Syncline, arrow shows plunge direction
Faults and Fissures With Offset
(Bar and Ball on downthrown side)
Fissure
Fissure Inferred
Scarp
Scarp Inferred
Cultural Features
Early Warning Rockfall Fence
River
10 foot Contour
Cross Section
Seeps
Prism Locations
Location of offset fence gate
Geologic Map prepared for the Colorado Department of Transportation
Geology by: Jonathan L. White and Jerry Higgins
Compiled by: Jason C. Wilson
Contour interval - 10 feet
150 0 150 300 450 600 FEET
COLORADO GEOLOGICAL SURVEY

Map Plate 1.

WEST LOST TRAIL CREEK STURZSTROM: A COMPOSITE LANDSLIDE

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Key Terms: composite landslide, landslide mapping, sturzstrom, San Juan Mountains

ABSTRACT

On July 30, 1991 a composite landslide containing approximately $8x10^6$ m^3 occurred in the San Juan Mountains of southwestern Colorado between elevations 12,394 ft (3778 m) and 11,000 ft (3353 m). Debris emanating from the lower elevations of this slide flowed into the West Lost Trail Creek valley damming West Lost Trail Creek. The purpose of this study was to map the landslide area and ascertain what type of landslide occurred and to verify if it was a "sturzstrom" rock fall avalanche. Pre and post-landslide topography and aerial photographs were evaluated as part of the study and a 25 mi^2 (64.75 km^2) are around the 1991 landslide was mapped and a landslide inventory complied, using bedrock, geology, aerial photos, topographic expression and Digital Elevation Models overlain by Digital Raster Graphic files. The authors identified much larger prehistoric sturzstroms close to the 1991 event, which also blocked West Lost Trail Creek. Four types of movement were identified within the 1991 landslide area: dilated translational blocks, retrogressive slump blocks, a large rock flow lobe and a chaotic debris field typical of sturzstroms. The authors also used Heim's Energy Line Modeling method to compare the rockfall avalanche portion of the landslide with previously documented sturzstroms. The article concludes with insights on the likely future behavior of the debris dam blocking the stream channel, which could impact recreational facilities and the Rio Grande Reservoir located a few miles downstream.

INTRODUCTION

The San Juan Mountains are widely recognized for having spawned innumerable landslides (Howe, 1909). After the Wisconsin-age glaciers retreated, stream erosion and mass wasting promoted the movement of great volumes of rock material down the canyon walls (Atwood, 1918). Most of the region's slopes were over-steepened by glacial excavation and have subsequently become involved in massive land slippage, which typifies much of the San Juan Mountain range.

This area of the San Juan Mountains is typified by large slope failures because the massive volcanic sequences contain layers of altered tuff or breccia that have weathered to smectitic clay

(Atwood and Mather, 1932), which exhibit low shear strength. The San Juan Tuff (Tev) appears to weather in this manner, promoting wide-scale land slippage throughout the range. Repeated glacial excavation of the West Lost Trail Creek valley caused slopes to be over steepened, which exacerbates mass wastage.

On July 30, 1991, a massive landslide occurred along the north face of Pole Creek Mountain in the San Juan Mountains in Hinsdale County, Colorado. The landslide is located along the border between sections 13 and 14, T.1 N. R. 5 W. or at approximately 37°47’30” N, 107°22’30” W (Figure 1). The failure appears to have originated at an elevation of around 12,394 ft (3778 m) above sea level. Most of the landslide mass translated more than 1,740 ft (530 m) to the valley below, rising as much as 197 ft (60 m) up the opposing slope (Figures 2 and 6). In addition to advancing up the opposing slope, two lobes of landslide material spread laterally both up and down the valley. The down valley lobe traveled approximately 1,400 ft (427 m) down the canyon bottom, while the up-valley lobe traveled about 200 ft (61m). As a result, West Lost Trail Creek was dammed, creating a small lake. Figures 2 and 6 show the areal extent of the landslide and the lake created by it. Due to the coarse nature of the debris, water has been able to percolate through the landslide mass, preventing the lake from achieving significant volume or overtopping the dam. The width of the main landslide mass varies between 1,400 ft (427 m) to 2,100 ft (640 m), and the maximum runout length was about 3,800 ft (1158 m). An eyewitness account (Rogers et al., 1999) indicates that the landslide moved more than one kilometer in 25-30 seconds, an average velocity of about 115 ft/sec (35 m/sec).

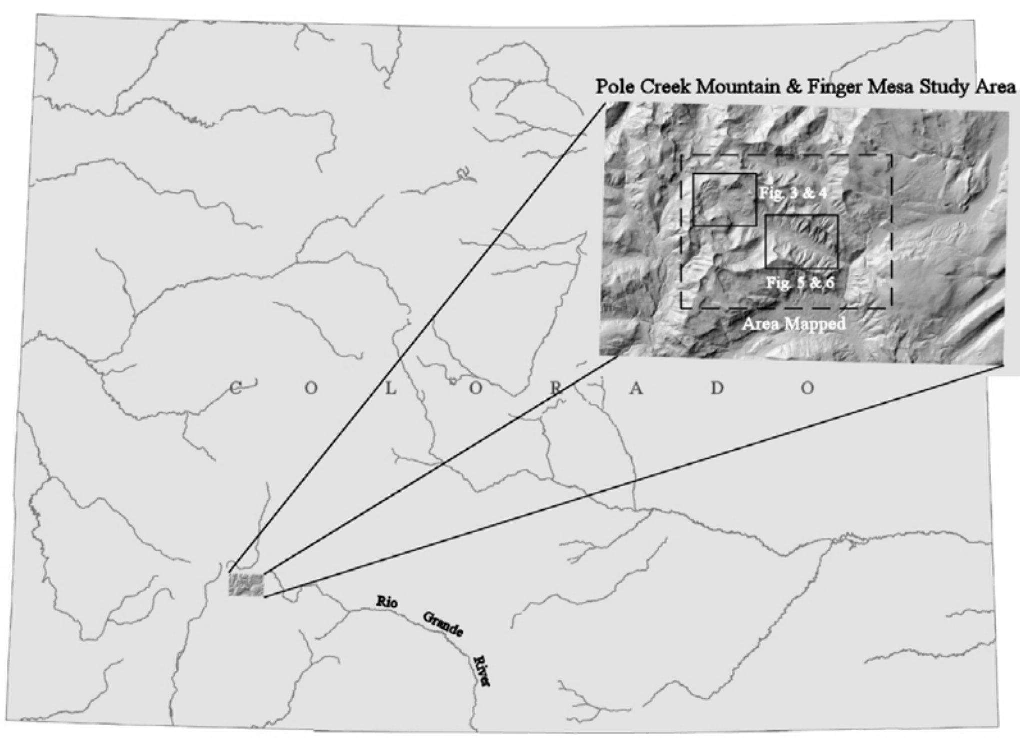

Figure 1. Location of the Pole Creek Mountain and Finger Mesa quadrangles in the San Juan Mountains of southwestern Colorado. The 1991 West Lost Trail Creek landslide occurred along the boundary between these quadrangles.

Figure 2. Aerial oblique view of the July 30, 1991 West Lost Trail Creek landslide (photo courtesy of Dave Noe)

Most of the landslide debris appears to consist of Tertiary volcanics, with minor amounts of late Wisconsin stage glacial till caught by the leading edge of the landslide and lifted with the advancing debris front. Post-failure reconnaissance of the landslide suggests that the movement began as a large block slide and quickly transformed into a rock fall avalanche, or “sturzstrom” (Rogers et al., 1999). The purpose of this study was to analyze the West Lost Trail Creek landslide, determine if it could be classified as a sturzstrom, and determine if it poses any future threat to people recreating in the area.

SITE GEOLOGY

The West Lost Trail Creek area is underlain by three mapped units. Cross and Larsen (1935) and Larsen and Cross (1956) identified these as the San Juan Tuff (Tsj), Treasure Mountain quartz latite (Ttu) and Sheep Mountain andesite (Tsm), of mid-to-late Miocene age. More recent work by Steven et al., (1974) has refined these to include a volcaniclastic facies of early intermediate lavas and breccias (correlative with the San Juan Tuff), the Eureka Tuff and the Henson and Burns Formations of the Silverton volcanic series, of Oligocene age. Previous workers have also identified glacial drift (Qd) in the valley floor. This article uses the unit designations suggested by Steven et al., (1974).

Steven et al., (1974) described the volcaniclastic series as “mostly reworked, bedded conglomerates, sandstones, and mudflow breccias of dark andesite and rhyodacite clasts.” More recently, Luedke and Burbank (1996) have described the San Juan Tuff as being thick- bedded to massive, gray to greenish gray, locally red or purple, reworked lahar and mudflow breccia with sandy tuff and tuff conglomerate. In this area slope failures often occur when massive volcanic rocks overlie poorly consolidated tuff or breccia, such as the volcaniclastic series because the underlying tuff weathers into smectitic clay and has a tendency to creep, slide, or flow. In the subject area, the volcaniclastic series overlies Precambrian-age igneous rocks.

The Silverton Volcanic Series overlies the volcaniclastic series. It is a collection of lavas, tuffs, and agglomerates named for the type section in the Silverton Quadrangle. They are subject to the same slope stability issues commonly associated with the volcanoclastic facies/San Juan Tuff (Ives and Bovist, 1978). The members of the Silverton volcanics present in West Lost Trail Creek canyon are the Burns Quartz Latite (Theb) and the Eureka Tuff (Tsd).

Glaciation

According to Atwood and Mather (1932), three distinct sequences of alpine glaciation swept through the steep sided canyons of the San Juan Mountains during late Pleistocene time, which they named the Cerro, Durango, and Wisconsin glacial periods. Cirques were formed centrally in the mountain region above the ice fields; these vary from a fraction of a square mile to as much as 15 mi^2 (38.9 km^2). The glaciers proceeded radially away from these catchment basins towards the bordering plateaus, scouring each canyon into the classic U-shape.

During Wisconsin time (100 to 110 ka), the Rio Grande Valley glacier occupied the eastern slope of the San Juan Mountains covering an area of more than 375 mi^2 (972 km^2) with a length of over 30 mi (48.3 km). The Lost Trail and West Lost Trail Creek valleys provided paths for tributary glaciers, which flowed into the much larger Rio Grande Valley glacier. These glaciers topped the margins of the valley and buried most of the neighboring divides except for the highest ones, reaching elevations over 12,000 ft (3659 m). In West Lost Creek Trail valley the glacier was over 1000 ft (328 m) thick. Although glacial till from the last three glacial sequences has been identified in the range, most of the till blanketing the floor of West Lost Trail Creek canyon is believed to have originated from the Wisconsin glaciation.

LANDSLIDE MAPPING

We contracted with the U.S. Forest Service GIS Laboratory to prepare 10 m Digital Elevation Models (DEMs) of the Pole Creek Mountain and Finger Mesa quadrangles, based on aerial photos imaged in September 1998, after the West Lost Trail Creek landslide occurred. The 32.8 ft (10 m) DEMs provided us with an additional tool to evaluate land slippage in the surrounding area and included post-failure contours of the 1991 landslide, with 16.4 ft (5 m) and 32.8 ft. (10 m) contour intervals.

The 10 m DEMs were useful for recognizing large landslides in the area. Figure 3 shows an example of a large slump-flow landslide complex located about 1.75 miles (2.9 km) upstream from the 1991 landslide. Note how individual blocks within the slump-flow complex, greater than 328 ft (100 m) across, are easily recognized. Figure 4 shows the same area with mapped landslide hazards, overlying the DEMs on Digital Raster Graphic (DRG) files of the same quadrangles. The DEMs have been manipulated to create a "hill shade" using the 2D Analyst

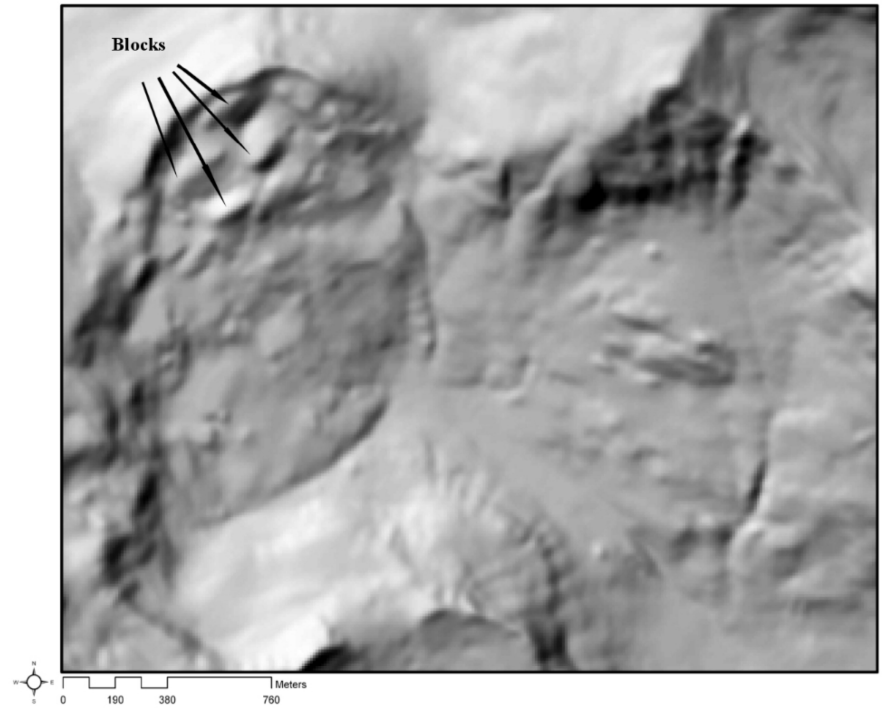

Figure 3. Portion of the 10 m DEM of the head of West Lost Trail Creek valley, about 2.9 km upstream of the 1991 West Lost Trail Creek landslide. Note that blocks greater than 100 m wide are easily discerned (arrows).

Figure 4. Landslide inventory map of the same area shown in Figure 3. This map overlays a shaded DEM, topography from a DRG, and mapped landslide contacts.

subroutine of ESRI's ArcGIS 8.2 software. This procedure provides a topographic base map with shaded relief from various azimuths, to favor recognition of landslide terrain, typically through back-shading. The landslide inventory map we prepared (Beckmann, 2002) covered an area of 25 mi^2 (65 km^2) at a scale of 1:12,000, too large to reproduce here.

The landslide map was intended to highlight features thought to be diagnostic of past land slippage, based on topographic expression and evaluation of stereopair aerial photos. A field reconnaissance was carried out in lower Lost Trail Creek and West Lost Trail Creek Valley, but the entire map area was not field checked.

Figure 5 shows a portion of the landslide inventory map in vicinity of the 1991 West Lost Trail Creek landslide. This mapping was based on pre-1991 topography, taken from the Pole Creek Mountain and Finger Mesa 7.5-minute quadrangles. Figure 6 shows the same area as Figure 5, on the 32.8 ft (10 m) DEM. The areal limits of the 1991 West Lost Trail Creek Landslide are clearly shown. The 8.33 ft (5 m) contours were rendered by using 3D Analyst subroutine of ArcGIS 8.2.

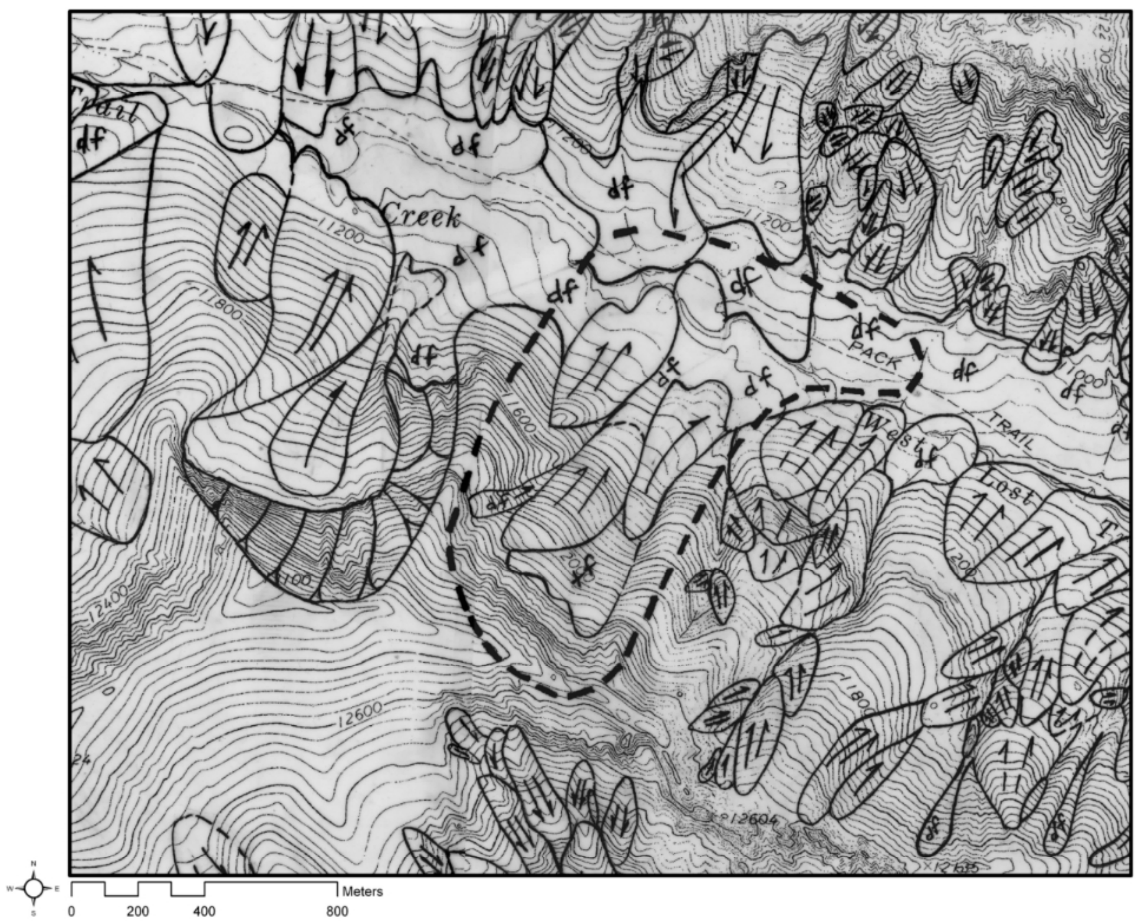

Figure 5. Landslide inventory map of the West Lost Trail Creek landslide area before the 1991 failure occurred. The heavy dashed line surrounds the 1991 landslide. Note the landslides which pre-existed the 1991 failure. The annotations "df" refer to debris fans and "tf" to talus fans", as introduced by Ives and Bovist (1978).

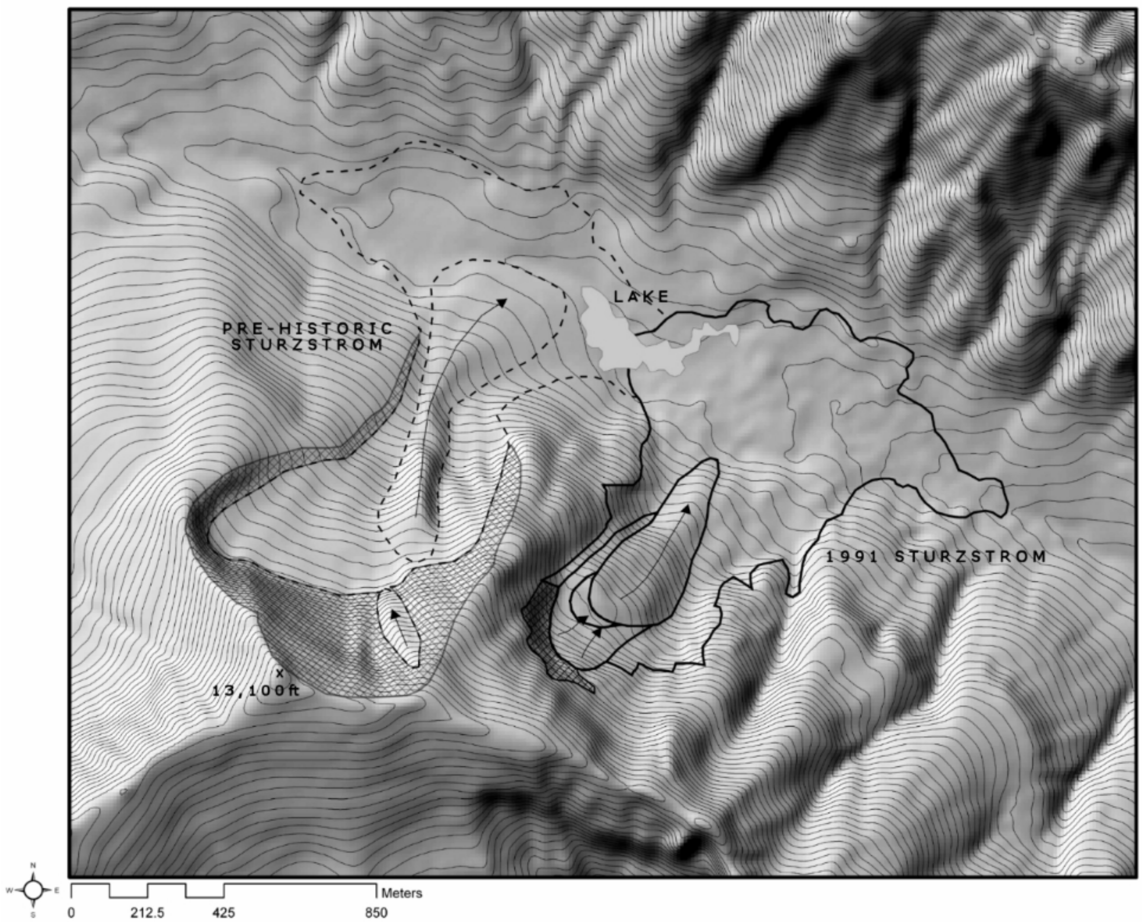

Figure 6. Present topography of the West Lost Trail Creek landslide area, denoting the lake created by blockage of the creek, as viewed on shaded 10 m DEM, with 10 m contours. The toe of the landslide swept across the valley floor, excavating glacial till and coming to rest with a depth of 5 to 15 m on the opposing valley bottom. A much larger prehistoric sturztsrom landslide (shown dashed) spilled off the north slope of Pole Creek Mountain just west of the 1991 landslide, blocking the channel in a similar manner.

In this part of the San Juan Mountains most of the landslides were found to be translational or rotational movements with earth flowage of the toes, that exhibit geomorphic expression indicative of deep-seated movement along discrete planar or curvilinear rupture surfaces. These rupture surfaces extend into the Tertiary volcanic rocks (bedrock) and may locally be associated with faults, joints, bedding planes, or other discontinuities. The more recent slides typically exhibit large blocks of source rock that have moved downslope, but older slides can be poorly defined. What begin as relatively intact block landslides often appear to disaggregate into flow slides, talus fans (tf) and debris fans (df) of Ives and Bovist (1978). These are usually shallower than translational slides and translate farther. Their basal slip surfaces appear broadly curvilinear. These flow fans often appear to incorporate older slide deposits, soil/colluvium, glacial detritus and badly weathered bedrock.

DESCRIPTION OF THE 1991 LANDSLIDE

The 1991 West Lost Trail Creek landslide occurred on the northeast flank of Pole Creek Mountain and incorporated Tertiary volcanics of the Burns (Theb), Eureka (Tsd) and the volcanoclastic facies of the San Juan (Tev) Formation. The apex of the landslide's scarp is located nearly 12,400 ft (3780 m) above sea level, in the Burns Formation. Originating as a block rockslide, the lower portion of the mass quickly disaggregated as it moved down slope, plowing approximately 20 ft of glacial drift and alluvial deposits up onto the opposing valley side. The landslide's kinetic energy was sufficient to cover the valley floor and advance as much as 80 ft (24 m) up the opposing valley slope. The final surface of the debris field is approximately 26 ft (8 m) above the valley floor. Rogers et al., (1999) estimated the landslide's surface area to be about 286 acres (1.15 km^2) with a volume of $10.5x10^6$ yds^3 (8×10^6 m^3).

The pre-1991 topography is suggestive of at least four smaller landslides pre-existed the 1991 failure, likely for the reasons stated above. Figure 7 presents a summary of annual precipitation for 1990 and 1991. Both of these were above average years for precipitation, following succeeding years of below-average precipitation (Western Regional Climate Center, 2002). Absorption of free water likely increased the overburden weight, creating additional driving force to the landslide and reducing effective stresses along the old rupture surfaces.

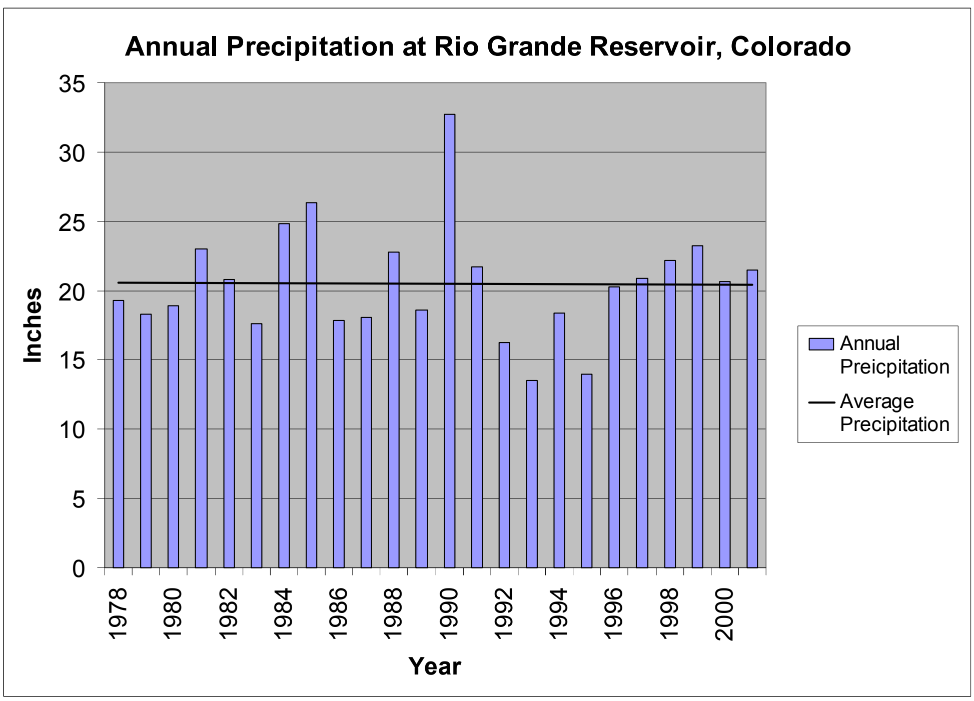

Figure 7. Annual Precipitation at the lower end of Rio Grande Reservoir (at the Farmers' Union Dam), about 10 miles downstream of the 1991 landslide. Note the extra normal rainfall that occurred in 1990, the year preceding the landslide.

The 1991 landslide appears to have been a composite landslide, wherein different types of landslide movements occurred simultaneously (Cruden and Varnes, 1997). Our landslide mapping and the eyewitness account and photos contained in Rogers et al. (1999) suggest that the landslide initiated as a series of retrogressive rotational rock slumps, which began to disaggregate with increasing translation down slope. The onset of movement is sketched in Figure 8.

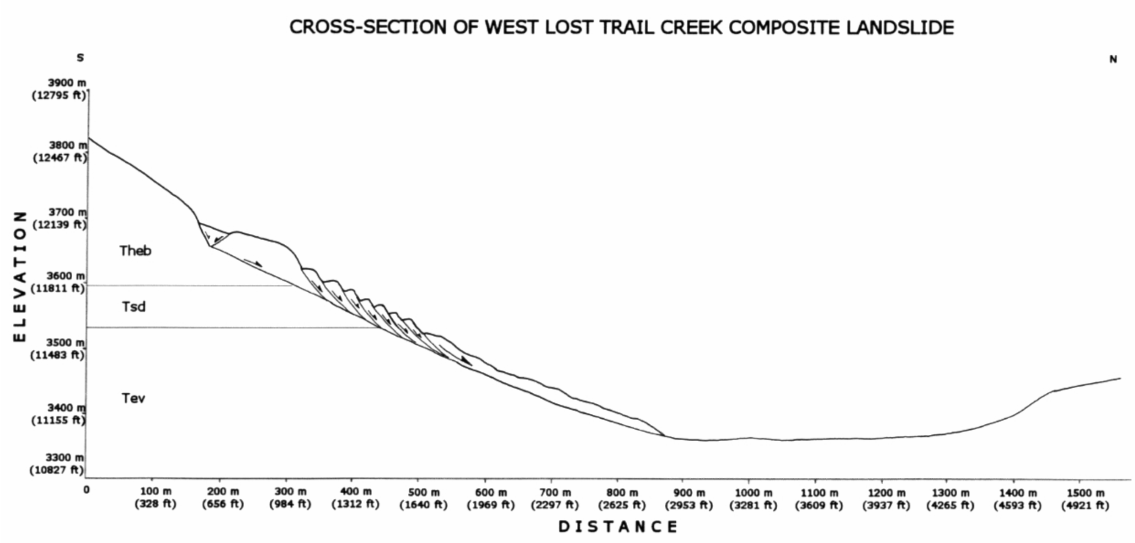

Figure 8. The 1991 West Lost Trail Creek landslide appears to have initiated as a series of retrogressive rotational rock slumps, progressing upslope over a 10 to 15-minute period (Rogers et al., 1999). The material below el. 11, 975 ft (3650 m) had previously been involved in land slippage.

The landslide debris downhill of the slump blocks had been involved in prehistoric landslippage and was likely dilated and disaggregated. This material began to flow as a semi-coherent rock slide, possibly being pushed from above by the en-echelon series of slump blocks. At the bottom of the slope the rock slide penetrated the glacial till filling the valley floor and began excavating the upper part of this unit, as sketched in Figure 9.

The landslide mass appears to have translated with increasing velocity as more material was incorporated into the moving mass. This description agrees with many eye witness accounts of sturzstroms (Skerner, 1989). Somewhere near elevation 11,483 ft (3500 m), complete detachment of the rock slide/rock flow occurred and material traveling further down slope accelerated across the valley floor as a sturzstrom rock avalanche. This transition is sketched in Figure 10. The sturzstrom debris traveled approximately 530 m along the fall line of the original landslide, not counting lateral movement up and down canyon.

Figure 11 shows the various components of the landslide area as it presently exists. A chaotic debris field left by the sturzstrom occupies the lower slope, from a low of about elevation 11200 ft (3414 m) up to about el. 11,650 ft (3551 m). The prominent lobe of a rock flow lies in the center of the landslide area between el. 3500 m and 3414 m. This is shown within a solid elliptical-shaped zone with a single arrow on Figure 6. The material within this flow is completely disaggregated, but did not detach catastrophically as part of the sturzstrom. A scarp-like detachment zone is easily observed in the transition between the lobe of rock flow material and the sturzstrom debris field, as sketched in Figure 11.

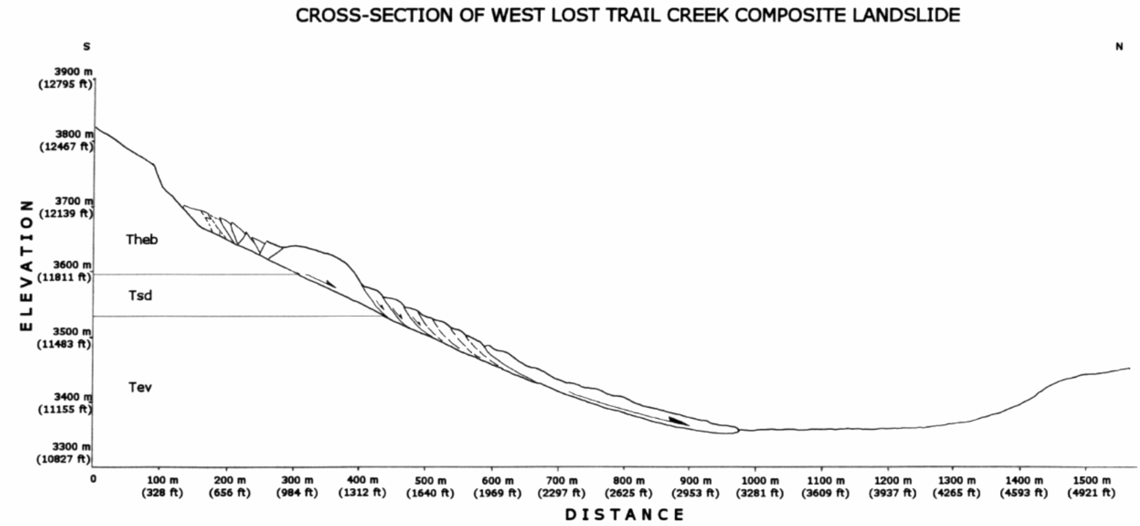

Figure 9. After the slumps joined together in a series of en-echelon blocks, the entire mass mobilized downslope, with an enormous rock flow progressively detaching from the upper mass and moving downslope onto the valley floor.

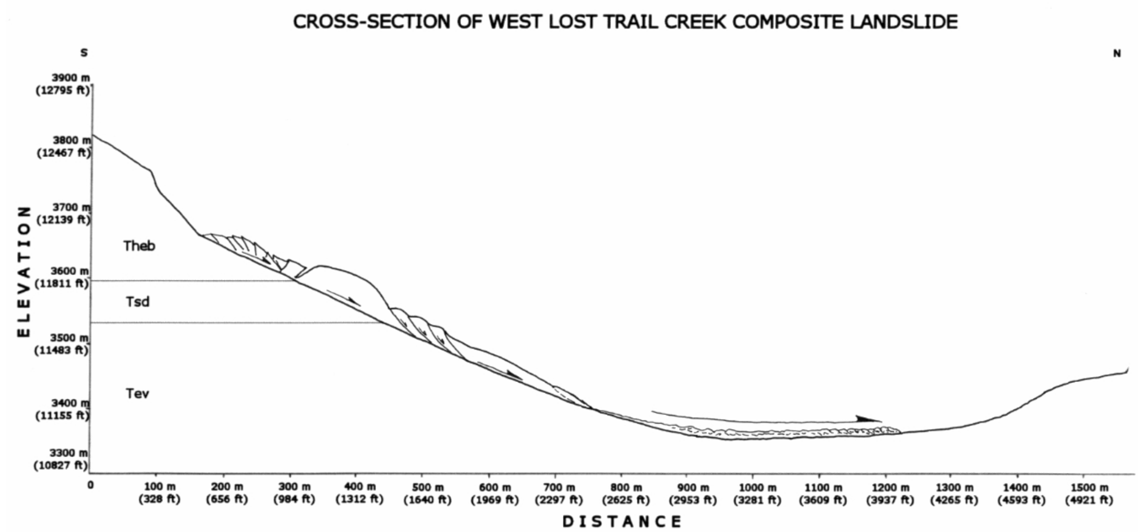

Figure 10. The rock flow emanating from the lower slope soon detached itself and speed across the valley floor as a sturzstrom, moving about 35 m/sec (Rogers, et al., 1999). A much greater portion of the landslide translated 100 to 300 m, but remained on the slope.

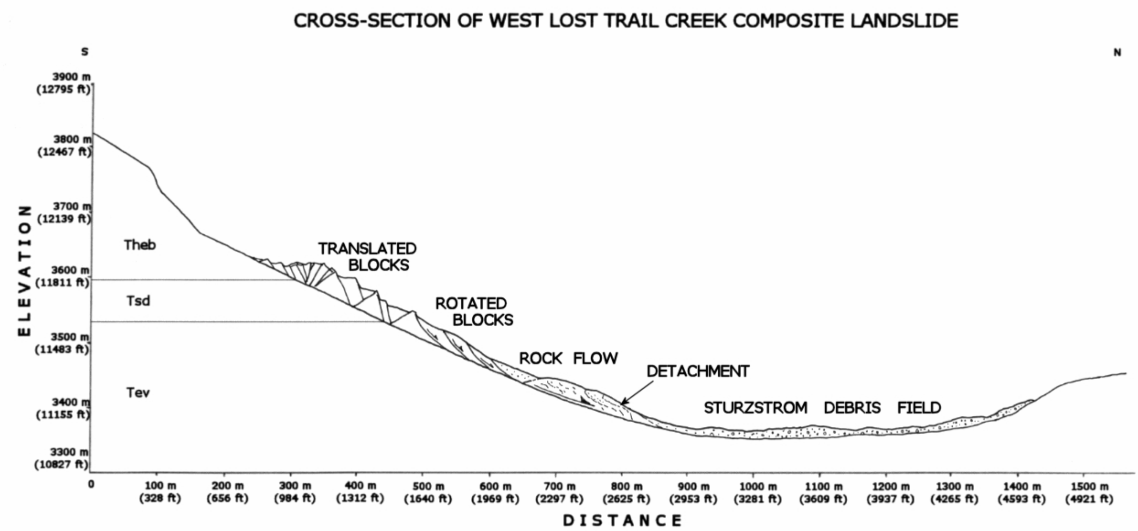

Figure 11. Cross section showing the 1991 landslide after movement ceased. The landslide deposits upslope of the sturzstrom debris field consist of a semi-coherent rock flow lobe, back-rotated rock slump blocks, and a series of translated blocks.

Above the rock flow materials the slide debris is increasingly blocky, suggestive of an en-echelon series of back-rotated rock slumps, which were co-mingled as they translated downslope. Above this is a series of very large rock blocks which have broken apart in a series of horsts and grabens as they were translated en-mass downslope about 660 ft (201 m). A conspicuous bench occupies the headscarp evacuation zone, which is filled with back-rotated blocks and talus belonging to the Burns quartz latite (unit Theb on Figure 11). The various rock slide styles are noted on Figure 11, extending through the center of the mapped blocks shown in Figure 6. We should note that rock material from either side (and higher elevations) of this "central" area became disaggregated at higher elevations and incorporated in both late-sequence rock flows and portions of the sturzstrom debris field.

The great majority of the sturzstrom debris field is comprised of San Juan Tuff, from the lower slope. Clast size within the debris field varied from house-size boulders to small pebbles (Figure 12). A small lake was impounded by the blockage of West Lost Trail Creek. However, the porous nature of the material allows water to percolate through the landslide dam. Isolated pools of water are scattered within the debris field above the former course of the creek (Figure 13).

STURZSTROMS AND HEIM'S ENERGY LINE MODEL

A sturzstrom is a particular type of rock fall avalanche which becomes fluidized as it falls, spreading itself like a debris flow over flat surfaces and traveling greater distances than are commonly observed or expected from sliding blocks using simple Coulomb friction models. The long runout distances observed in sturzstroms appears to a function of their great volume, between 10^5 and 10^{11} m^3 for terrestrial landslides and up to 10^{13} m^3 for subaqueous landslides.

Figure 12. The variability in size of the landslide debris can be appreciated in this view of the debris field, looking upstream (westerly) towards the impounded lake shown in Figure 6. Many of the boulders are as large as small buildings (25 x 50 ft or more).

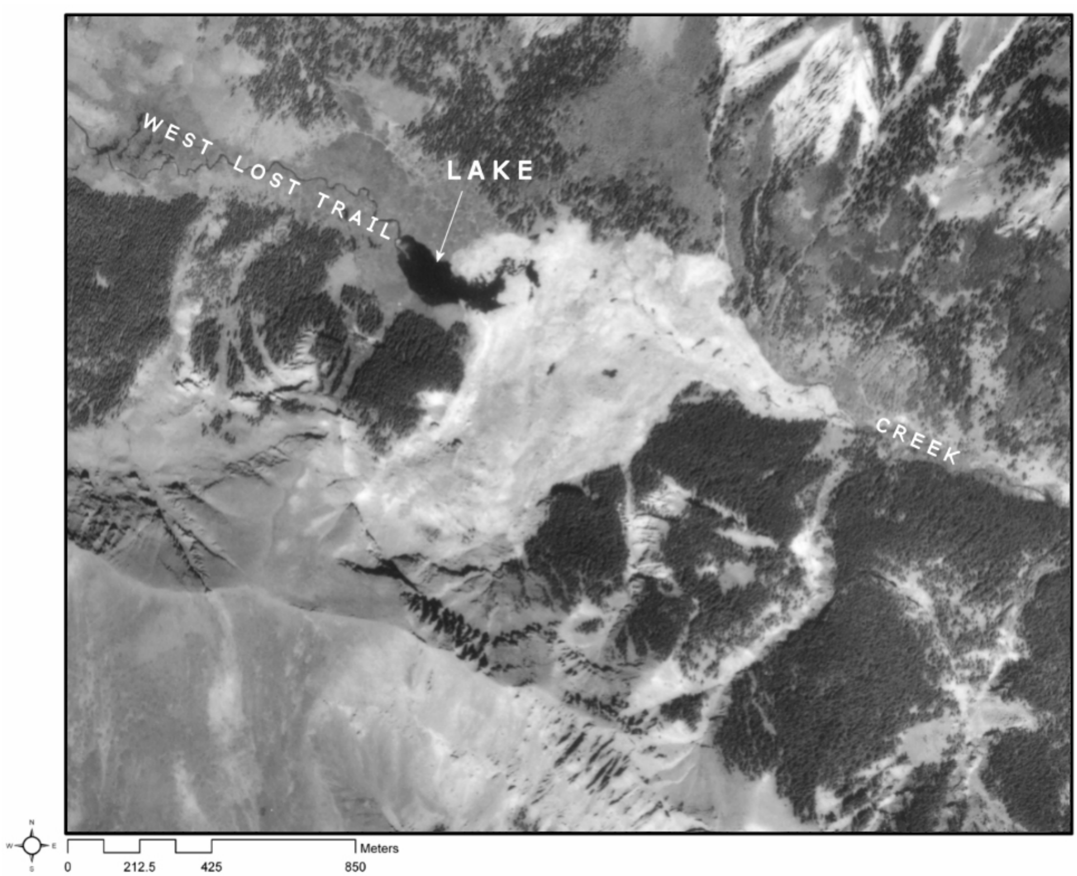

Figure 13. Aerial photo of slide imaged in September 1998 showing main impoundment (upper left center) and smaller ponds distributed about the debris field. The creek discharges through the 1991 debris field and passes on downstream.

Sturtzstroms often begin as rock falls, but as movement continues, the landslide fragments into a wide range of angular particles ranging from pebble to boulder size. When fully mobilized, surtzstroms behave like fluids and their debris translates much farther downslope than can be explained using conventional Coulomb friction models for landsliding.

The Swiss geologist Albert Heim (1849-1937) conducted the first notable work describing sturzstroms in his book *Der Bergsturtz von Elm*, describing the Elm landslide of 1881 (Heim, 1882). Heim believed that three main factors were needed to trigger rockslides: 1) a large volume of material, 2) some initial fall height, and, 3) the regularity of the flow path (Heim, 1932). Legros (2002) have stated that the operative trigger fomenting sturzstroms is simply the mass of the initial detachment, citing a minimum volume of 10^5 m^3. This threshold agrees with observations of sturzstroms tabulated by others (Hsu, 1975, 1978; Keefer, 1984). Soon after the sliding block detaches, it quickly disintegrates into smaller angular components. Debris size varies throughout the slide mass and can range from rock dust to boulders the size of small buildings.

One of the most intriguing characteristics of a sturzstrom is how a dry heterogeneous mass can flow as a cohesionless fluid for great distances over gradients as low as 1°. Until recently, previous workers had generally assumed that some sort of interstitial fluid must be present to reduce the effective normal stresses between blocks. They suggested that the interstitial "fluid" might be compressed air, mud and water, or fine debris particles (Shreve, 1968; Hsu, 1975). More recently, others have suggested that the sturzstroms are characterized by large volumes of material which conserve momentum during their fall, and that the volume of slide debris controls and correlates with observed runout distance (Kilburn, 1998; Legros, 2002; Iverson, 2003).

In Heim's 1932 text his physicist colleague Eugen Müller-Bernet presented a model to estimate runout distance by considering physical traits of the landslide sites and back-analyzing the effective friction of the sliding surface. In this model, the path of a sturzstrom is divided into two basic sections: 1) a steep slope marking the detachment area, and 2) a flatter slope denoting the runout area (Figure 14). The two regions are separated by a "turning point", shown in Figure 14. As the rock slide detaches itself from the parent ridge, it travels along the "fall slope," which Heim described as being inclined between β_1= 25° and 89°. Heim believed that during the initial phase of sliding the majority of the rock mass's kinetic energy was accumulated. As the slide continued to translate downhill, the mass fragments and dilates, dissipating frictional energy as it rolls onto a flatter runout path, which Müller delineated as having an average slope inclined β_2 degrees from horizontal (an upward, or negative, slope on the opposing valley side could be accommodated in his model). This lower slope (β_2) varied from dead level to as much as 30°. Until recently, most workers assumed that kinetic energy was expended through frictional resistance along the base of the flow in the runout phase. Heim (1932) reasoned that the slide's volume and velocity decreased as the coefficient of basal friction and runout area increased, but his Coulomb friction model does not include mass or momentum conservation (Iverson, 2003). Hsu (1975) used ELM analyses to develop a range of expected values relating landslide volume, equivalent friction and runout which has proved useful for classifying sturzstroms with volumes up to 10^{11} m^3.

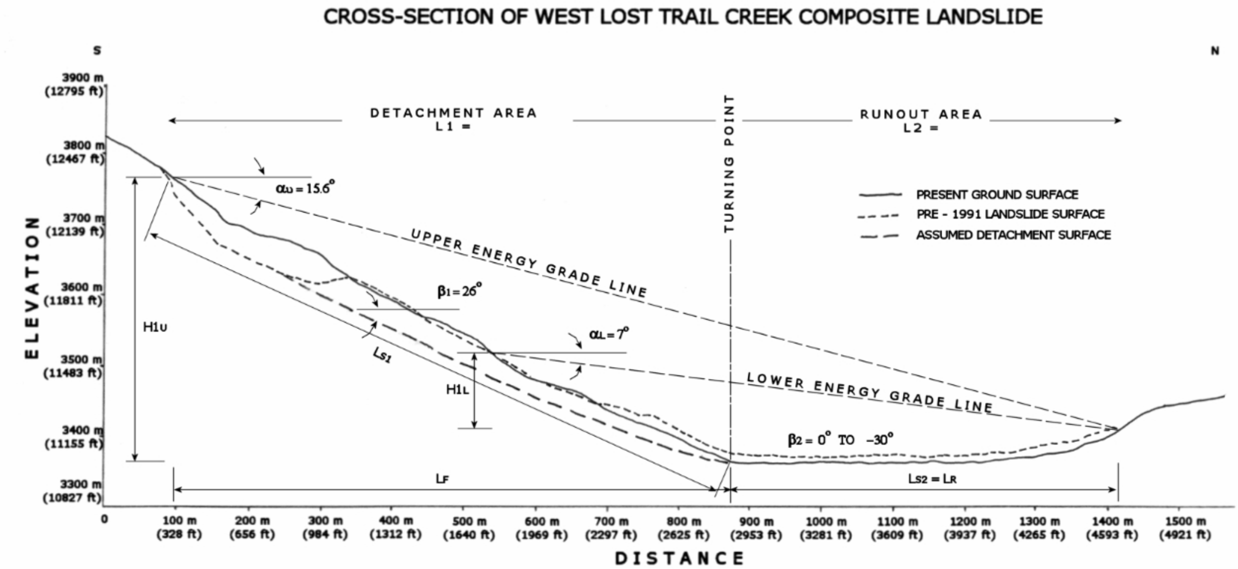

Figure 14. Physical aspects of the 1991 West Lost Trail Creek landslide using Heim and Müller's energy line model for evaluating the runout of a sturzstrom. We applied lower and upper bounds to model the runout, because complete detachment occurred below el. 11,480 ft (3500 m).

An approximation of the average frictional energy can be obtained from Müller's "energy grade line" (Figure 14). This is the angle of an arbitrary line drawn from the uppermost part of the slide scarp to the tip of the stationary rubble flow. Referred to as α, this angle has been observed to be in the range of 8° to 20° for most sturzstrom rock avalanches. Shreeve (1968) suggested using the tangent of this angle as an "equivalent coefficient of friction" along the slide path, a term retained by Hsu (1975, 1978) in his later work. Müller (in Heim, 1932) related this expenditure of energy through basal friction loss to the Energy Grade Line (EGL) used in hydraulics to describe the loss of energy with flow distance. As a consequence, their prediction of runout distance became $L = H/\tan ø$, where L is the runout distance, H is the fall height and ø is the angle of reach (α), or equivalent [or average] friction angle derived from the EGL plot shown in Figure 14. Data suggests that sturzstrom runout increases in proportion to the square root of their volume (Hsu, 1975; Kilburn, 1998).

ENERGY LINE MODEL ANALYSIS OF THE 1991 LANDSLIDE

The energy line model (ELM) analysis of Heim (1932) was performed on the 1991 West Lost Trail Creek landslide. ELM analysis can provide approximate values for an average coefficient of basal friction, average velocity, and runout distance, irrespective of mass and momentum conservation (Kilburn, 1998; Legros, 2002; Iverson, 2003). The predicted values were compared with what was actually measured on the post-failure map of the landslide area, prepared from aerial photos imaged in 1998.

Figure 14 presents a maximum cross section taken through the center of the 1991 landslide, based on both pre and post-landslide topography. A prominent headscarp evacuation zone lies between el. 11,870 ft (3618 m) and 12,394 ft (3778 m). The likely depth and inclination of the basal sip surface was inferred from the headscarp graben geometry using the technique suggested by Cruden, Thomson and Hoffman (1991). This appears to have been the initiation point of the 1991 slide event. There were also multiple benches visible on the cross section, which suggests smaller slope failures had occurred along much of this profile (Figure 5). The upper extent of Wisconsin glaciation on this cross section appears to have been somewhere close to 12,600 ft (3841 m) elevation.

The angle of reach (α) between the uppermost point of the scarp and the furthermost tip of the rubble stream on the opposing slope is approximately 15.6° from horizontal. This is shown as the Upper Energy Grade Line on Figure 14. The depth and position of the basal detachment surface for the block was estimated based on comparisons of pre and post-slide topography (Figures 5 and 6). The angle of this failure surface coincides to the angle of the fall slope (β_1) and is approximately 26° from horizontal. Some of the slide material ran up the opposing valley wall and stopped, but sufficient kinetic energy was retained to allow some of the slide debris to bank off the opposing valley side and run down canyon an additional 1,200 ft (366 m). The angle of the down canyon runout slope (β_2) varied from ½^o to -30^o (on the opposing valley side).

The ELM analysis employs energy relationships evolved from fluid mechanics, applying energy head in terms of elevation differential. The potential energy head of the slide mass is assumed equal to the height of the fall slope H1 (Figure 14). The frictional work (energy head loss) expended along the fall slope can be approximated by the relationship $W_f = (\tan\alpha)(\cos\beta_1)L1$, where W_f is the work expended, $\tan \alpha$ is the equivalent coefficient of friction, $L1\cos \beta_1$ is the normal component of the slide mass acting on the sliding surface, and L1 is the length of the initial fall slope.

The kinetic energy head of the slide mass (KE) gained by the fall is approximated by the formula $KE = (H1)W_{LS} - W_f$. The velocity of the slide mass at the base of the fall slope can be approximated using $V = \sqrt{2}\,(g)\,(KE)$, where g is the acceleration of gravity. The time elapsed traveling down the fall slope (t_1) can be approximated as $t_1 = L1/V$.

As the rock fall makes the transition from the fall slope to the runout slope it will begin to decelerate and the frictional expenditure along the more gradual slope is accounted for. If the runout slope is inclined, some potential energy ($H2 \times W_{LS}$) may actually be gained on the runout path. This energy head can be estimated by using $H2\,(W_{LS}) = L2\,(\sin\beta_2)$, where L2 is the length of the runout slope, β_2 is the slope of the runout surface. The deceleration along the runout slope, (c), is given by $c = (g)\,(\sin\beta_2 - \tan\alpha\,\cos\beta_2)$. The time for total stop to occur (t_2) after deceleration initiates is given by $t_2 = V/c$.

The total runout time is estimated by adding t_1 and t_2. The length of the sturzstrom runout is calculated by using fluid mechanics concepts of potential and kinetic energy head, adding the kinetic energy head at the base of the fall slope to the potential energy head gained on the runout slope and setting this sum equal to the frictional work expended along the runout path. This relationship can then be solved for the length of runout (L2). The mass of the landslide remains constant, so the relationship becomes: $KE + L2\,(\sin\beta_2) = (\tan\alpha)\,L2$.

We applied these concepts to two models: an upper-bound ELM analysis assuming the fall height to extend to the top of the landslide headscarp (el. 12,394 ft [3778 m]) and a lower-bound ELM, extending only from el. 11,483 ft (3500m). We believe the lower bound model more accurately portrays the kinematic situation governing the sturzstrom portion of the 1991 landslide event.

The ELM analysis assuming full slope height utilized the following values: L1=2559 ft (780 m), H1=1,345 ft (410 m), H2=0 m, β_1=26°, β_2=1/2°, and α=15.6°. The frictional work was found to be 643 ft (196 m). The kinetic energy head remaining after the fall is about 702 ft (214 m). The velocity at the base of slope was estimated as 65 m/sec. The time elapsed during initial fall should have been close to 12 sec. The potential energy head gained on the runout slope was negligible. The deceleration head loss upon runout was calculated as -8.7 ft/sec (-2.65 m/sec). The time for total stop was 24.5 sec and the total runout time estimated as being 36.5 sec. The length of runout was calculated to be 2,592 ft (790 m). The initial runout track across the valley was about 1,739 ft (530 m). If we include the down-valley runout, the runout distance was up to an additional 1,400 ft (427 m).

The lower-bound ELM analysis assumed detachment from the lower slope, initiating below elevation 11,480 ft (3500 m). This analysis utilized the following values: L1=1,083 ft (330 m), H1=525 ft (160 m), H2=0, β_1=24°, β_2=1/2° to -30°, and α=7°. The frictional work was found to be 121 ft (37 m). The kinetic energy head remaining after the fall was 220 ft (67 m). The velocity at the base of slope was estimated as 119 ft/sec (36.3 m/sec). The time elapsed during initial fall should have been close to 9 sec. The potential energy head gained on the runout slope was negligible. The deceleration head loss upon runout was calculated as -3.67 ft/sec (-1.12 m/sec). The time for total stop was 32.4 sec and the total runout time estimated as being 41.5 sec. The length of runout was calculated to be 1,926 ft (587 m). This was closer to the observed value of 1,739 ft (530 m) than that calculated assuming the full slope height.

The eyewitness account of David English (in Rogers et al., 1999) suggests that the slide traversed 3,280 feet in about 25-30 seconds. This would equate to an average velocity of about 129 ft/sec (39.3 m/sec); within 8 percent of the value predicted by the lower-bound ELM analysis. The eyewitness account also suggests that the landslide gradually accumulated the requisite mass and momentum for its lower portion to become a sturzstrom, gaining momentum and disaggregating as it moved downslope. This would account for the somewhat slower translation velocities than described by other sturztsroms.

The lower-bound ELM analysis appears to have predicted the observed traits of the 1991 landslide with remarkable accuracy. Legros (2002) stated that Heim's ELM analyses provide fairly accurate predictions for smaller scale rock avalanches and sturzstroms, but underestimates runout for landslides with volumes greater than about 1.3 million yds^3 (1 x 10^6 m^3). Heim's ELM model fails to account for mass, conservation of momentum or energy loss on turns, such as must have occurred when some of the slide material banked off the opposing slope and turned both up and downstream.

HAZARD EVALUATION

A major question facing the U.S. Forest Service managing the West Lost Trail Creek Wilderness is whether the West Lost Trail Creek landslide poses a future risk to recreational use of the area, as well as the operation of Rio Grande Reservoir, whose tailwater is just 5 mi (8 km) downstream. The landslide area exhibited topographic features characteristic of at least three or four landslide events prior to the July 1991 landslide (Figure 5). These appear to have been large translational blocks which moved between 10 and several hundred meters, but of insufficient volume and initial fall height to have triggered sturzstroms. One of the factors that allowed the West Lost Trail Creek slide to travel as far as it did was its large initial volume of 8×10^6 m^3. It is unlikely that a future landslide of significantly lesser volume would reach the valley bottom, let alone carry across the valley floor, which is about 1,600 ft (488 m) wide.

It is difficult to predict the likelihood of a catastrophic failure of the present landslide dam. There is abundant geomorphic evidence of older landslide dams along West Lost Trail Creek, immediately upstream of the 1991 landslide (Figure 6). The impact of past landslide dams can also be appreciated by examining the pre-1991 profile of West Lost Trail Creek, shown in Figure 15. The average hydraulic gradient is determined by dividing the fall of the channel over its course and adjusting the profile downward for accreting tributary watershed (Hack, 1973). This is the gradient we would expect to observe in a channel at equilibrium with its watershed (Hack, 1973). The stream has excavated a series of cascading rapids through the older landslide dams, which extend over a mile (1.6 km) upstream of the 1991 landslide. It appears that these older landslide dams are gradually being excavated by a low streampower watercourse, rather than catastrophic excavation by overtopping. Further evidence of this premise is the absence of inset terraces commonly associated with landslide dam outburst floods downstream of the landslide dam sites.

It is unlikely that West Lost Trail Creek will be able to excavate the landslide dam in the foreseeable future. The geomorphic response in this area is retarded by the low available streampower. The site is located only 2.5 mi (4 km) from the Continental Divide, with a tributary watershed of only 4.5 mi^2 (11.65 km^2). The lake impounded by the dam occupies 8 acres (32,000 m^2), with an unknown depth (likely less than 4 m). This would be an insufficient volume of water to excavate the 8 m deep debris field in broken rock.

Within the next few millennia the pond created by the 1991 slide will likely trap sufficient fine grained sediment to begin filling the pore spaces along its floor. As sedimentation occurs, interflow through the slide mass will be progressively retarded and the pond should enlarge until such a time it will intrude further onto the surface of the debris field. Eventually, the pond will reach an elevation sufficient to initiate overtopping of the debris field by overland flow. At this juncture, some runoff will probably percolate back into the angular slide debris, similar to what occurs at present, and discharging from the downstream toe of the debris field. If sufficient volume of water were to discharge from the toe of the debris field with excess hydraulic head, a process of headward erosion might be induced at the downstream transition between the undisturbed channel and the slide debris. Like the landslide dams upstream, a series of quasi-stable rapids would likely develop, not a catastrophic breach of the landslide dam.

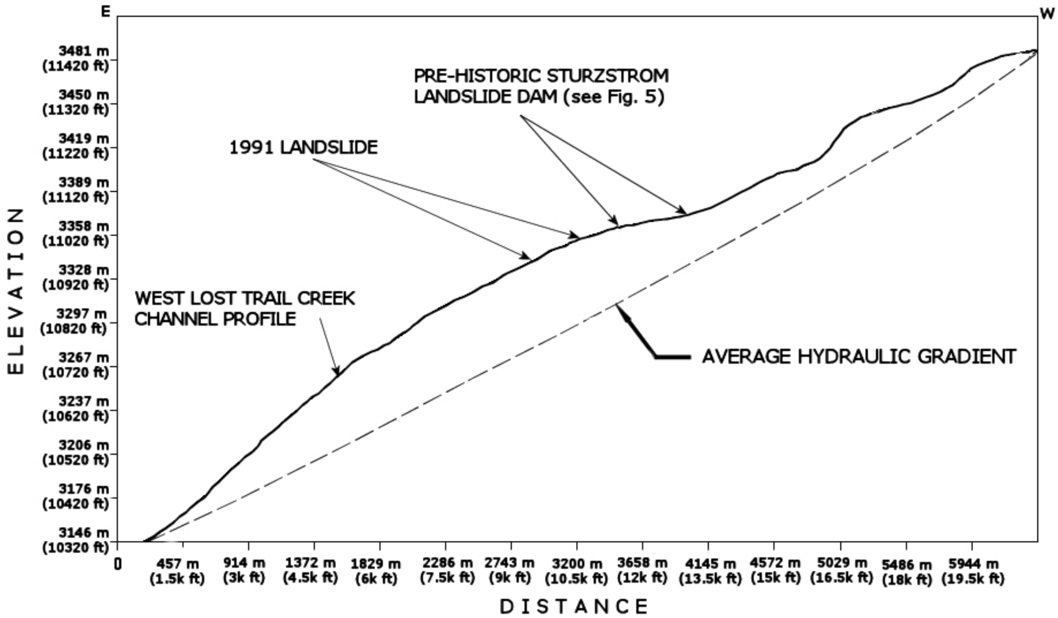

Figure 15. West Lost Trail Creek channel profile, between its confluence with Lost Trail Creek (point of origin, at left) and 2 mi (3.2 km) upstream of the 1991 landslide. The convex-upward profile is diagnostic of a channel containing much more debris than it has hydraulic capacity to remove. This depositional "plug" appears to have been deposited by repeated landslide dams spilling into the valley.

SUMMARY AND CONCLUSIONS

The 1991 West Lost Trail Creek landslide appears to be a composite landslide, which initiated movement as a retrogressive series of translational block landslides that rapidly disaggregated and evolved into a sturzstrom rockslide avalanche. The debris field at the toe of the landslide displays ample evidence of fluid-like flow, resulting from dynamic flow conditions which temporarily engendered negligible shear stress between individual blocks. This allowed the sliding mass to behave like a fluid. Evidence for fluid-like flowage in 1991 slide can be observed by the manner in which the slide turned and flowed, both up and down the valley floor, and by the flow features observed in the debris field (Figures 2 and 13). Although, because of their large volume, sturzstroms are relatively infrequent events, a much larger sturzstrom cascaded off the north face of Pole Creek Mountain (el. 13,100 ft/3993 m) and blocked the canyon immediately upstream of where the 1991 landslide occurred (Figure 6). This larger sturzstrom occurred sometime during the Holocene (last 11 ka), after the last glaciations receded.

An unusual aspect of the 1991 sturzstrom was the relatively thin (26 ft/8 m) and uniform layer of rock debris spread across the valley. The debris stream rose as much as 197 ft/60 m above West Lost Trail Creek, extending up the opposing valley side. This suggests a high fluid viscosity at the time of deposition. Heim (1932) theorized that sturzstroms appear to propel themselves through conservation of momentum, by transferring kinetic energy from the head of the slide towards the front, or toe. This concept remains widely accepted (Kilburn, 1998; Legros, 2002; Iverson, 2003). As the slide begins to dissipate kinetic energy on its runout leg, some intragranular shear strength is regained, much like an aqueous debris flow. Translation and disaggregation initially cause the rock mass to dilate. Spreading across the runout area decreases the ratio between slide (fluid) volume to base area, akin to the wetted perimeter to volume ratio in channel hydraulics. The decrease in volume to wetted perimeter should expend friction at an increasing rate, causing deceleration. This deceleration increases rapidly until the slide mass stops suddenly, seeming to "freeze in time". This sudden deceleration captures many fluid-like features, such as lateral flow ridges, compression ridges, en-echelon fissure fields and inverse sorting of entrained particles, similar to features observed in debris flow fans and some lava flows. A well defined terminal rim was observed at this site, which excavated valley bottom sediments and carried it up on the snout of the debris train. Very little rubble from the slide body was observed beyond this terminal rim.

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RESEARCH CONDUCTED AT THE SLUMGULLION EARTH FLOW, 1958 TO 2002

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Key Terms: landslide, earth flow, deformation, landslide monitoring

ABSTRACT

The Slumgullion earth flow, in the San Juan Mountains of southwestern Colorado, has attracted the attention of many investigators since the 19th century. The earth flow consists of a younger, active, upper part that moves on and over an older, much larger, inactive part. Displacement data have been collected on the active part of the earth flow during the past 40 years by surveying, photogrammetric, field-instrumentation, and GPS methods. These data reveal that annual surface velocities vary from 20-23 ft/yr (6-7 m/yr) in the narrowest region of the flow to 3-7 ft/yr (1-2 m/yr) in the head and toe regions. Recent work on the active flow shows that daily velocities vary seasonally and that accelerations and decelerations closely correlate with fluctuations in hydrologic input at the surface of the earth flow. This implies that pore pressures at the basal surface(s) of the earth flow respond quickly to moisture flux at the earth flow's ground surface. Finally, leveling surveys indicate that the inactive part of the Slumgullion earth flow is deforming by a general depression of the ground in front of the advancing toe.

INTRODUCTION

The Slumgullion earth flow, in the San Juan Mountains of southwestern Colorado (Figure 1), has attracted the attention of many investigators. From the 19th century to the middle of the 20th century these include Endlich (1876), Cross (1909), Howe (1909), Larsen (1913), Atwood and Mather (1932), and Burbank (1947). The first detailed investigation of the history and rates of movement of the Slumgullion earth flow was performed by the U.S. Geological Survey (USGS) from 1958 to 1961 (Crandell & Varnes, 1961). In 1990, the USGS, with the assistance of scientists provided by a cooperative agreement between the USGS and the Italian National Research Council (CNR), began a study of the Slumgullion earth flow that included detailed mapping, determinations of earth-flow kinematics by precise surveying, leveling, and photogrammetric methods, geophysical investigations, and efforts to establish the history of movement. Results were published in several USGS open-file reports, in a USGS Bulletin (Varnes & Savage, 1996), as a USGS map (Fleming et al., 1999), and as a field-trip guidebook (Fleming et al., 1996). From 1998 to 2002, the USGS and Brigham Young University (BYU) conducted a NASA-funded study to measure seasonal movement of the active part of the Slumgullion earth flow using Global Positioning System (GPS) surveys, extensometers, and an airborne Synthetic Aperture Radar (SAR) system. Measured movements were correlated with simultaneously measured temperature, rainfall, snow depth, and ground- and surface-water levels and pressures (Coe et al., 2003). In this paper, we describe the geographic and geologic setting and the general characteristics of the earth flow and review results from Slumgullion research conducted since 1958.

LANDSLIDE TERMINOLOGY USED IN THIS PAPER

The Slumgullion landslide is a *complex* or *composite* landslide in which different types of movement occur in a variety of volcanic materials, ranging from rock falls at the main scarp to earth flows and earth and debris slides throughout the landslide. However, most of the Slumgullion landslide fits the Varnes (1978) description of an earth flow, which it has been known as historically (Crandell & Varnes, 1961). For these reasons and for ease of presentation, we will refer to the Slumgullion landslide throughout this paper as an earth flow.

GEOGRAPHIC AND GEOLOGIC SETTING OF THE SLUMGULLION EARTH FLOW

The Slumgullion earth flow occupies a valley originally formed by Slumgullion Creek, a tributary of the Lake Fork of the Gunnison River (Figure 1). The earth flow formed as a result of the collapse of hydrothermally altered Tertiary volcanic materials on the south edge of Mesa Seco (Figure 2). The detached materials slid and flowed downhill damming the Lake Fork of the Gunnison River, impounding 2.0 mile (3.3 km) long Lake San Cristobal (Schuster, 1996; Figure 2). The earth flow now consists of a younger, active, upper part that moves on and over an older, much larger, inactive part. The active part of the earth flow is 2.4 mi (3.9 km) long, covers an area of 0.56 square mi (1.46 km^2), and has an approximate volume of 26 x 10^6 cubic yds (20 x 10^6 m^3), based on an estimated average thickness of about 33 ft (10 m) (Parise & Guzzi, 1992). The entire earth flow is 4.2 mi (6.8 km) long, covers an area of 1.80 square mi (4.74 km^2), and has an estimated volume of about 222 x 10^6 cubic yds (170 x 10^6 m^3) (Parise & Guzzi, 1992). Total relief from the toe of the inactive part at the Lake Fork of the Gunnison River to the top of the 820 ft (250 m) high main scarp is about 3,280 ft (1,000 m). Part of the inactive toe lies beneath the waters of Lake San Cristobal and the morphology and character of this part of the earth flow are poorly known. Based on the results of seismic reflection and refraction profiles, Williams & Pratt (1996) estimated a maximum thickness of about 312 ft (95 m) for the inactive earth flow along a profile downslope from the active toe and east of Colorado State Highway 149 (Figure 3). By using geomorphic evidence, Parise & Guzzi (1992) estimated the thickness of the inactive earth flow to be 394 ft (120 m) in the same area.

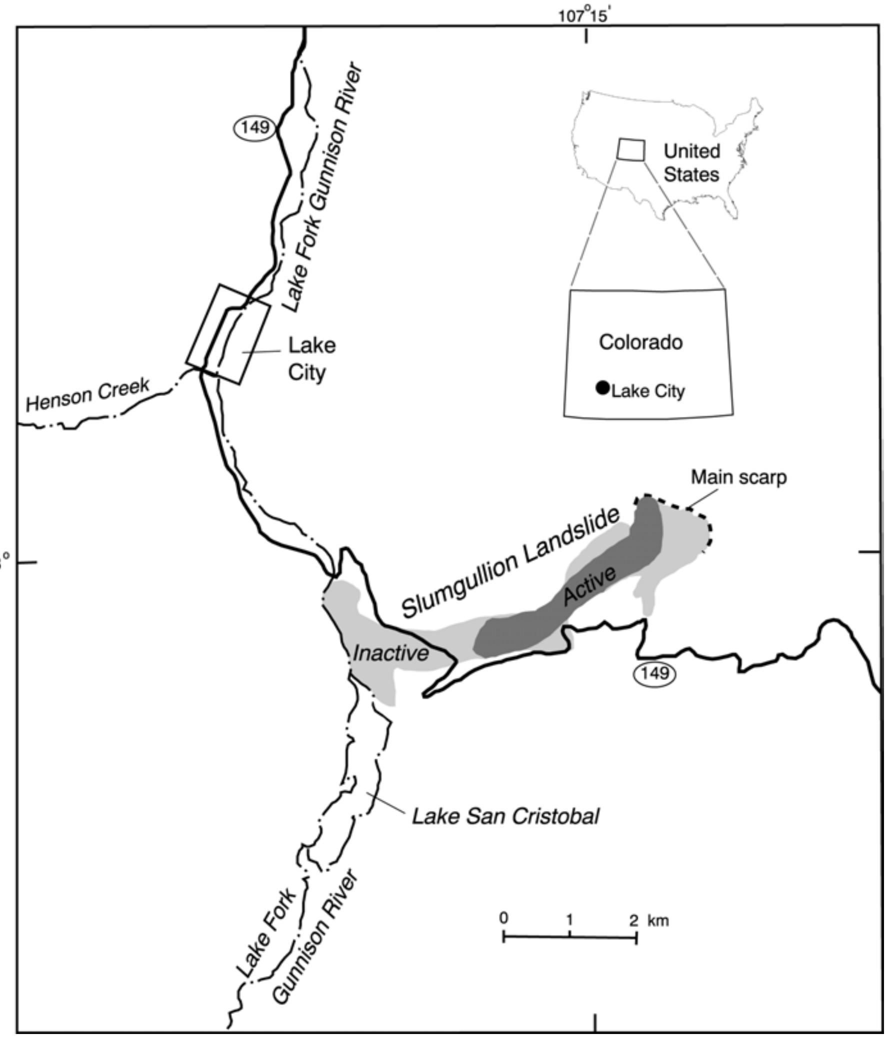

Figure 1. Map showing the active and inactive parts of the Slumgullion earth flow. Colorado State Highway 149 is also shown.

Rocks exposed in the main scarp are hydrothermally altered and unaltered volcanics that include tuffs and intrusive and extrusive rocks of variable compositions (Diehl & Schuster, 1996). Hydrothermal brecciation and alteration, combined with further weakening of the rock mass by intersections of numerous faults in the main scarp area probably played a key role in controlling the location and initiation of the earth flow. As a consequence of the extensive alteration and faulting in the main scarp (source) area, the matrix of the earth flow is composed of fine sands, silts, and clays. Large assemblages of unaltered volcanic boulders are also locally present in and on the earth flow.

Several episodes of earth-flow movement have been identified based on radiocarbon dating, differences in degree of weathering of earth-flow materials, depth of soil formation, and surface morphology (Crandell & Varnes, 1961; Chleborad, 1993, 1996; Madole, 1996; Fleming et al., 1999). The first episode blocked Slumgullion Creek 1,000-1,300 years ago; a second episode

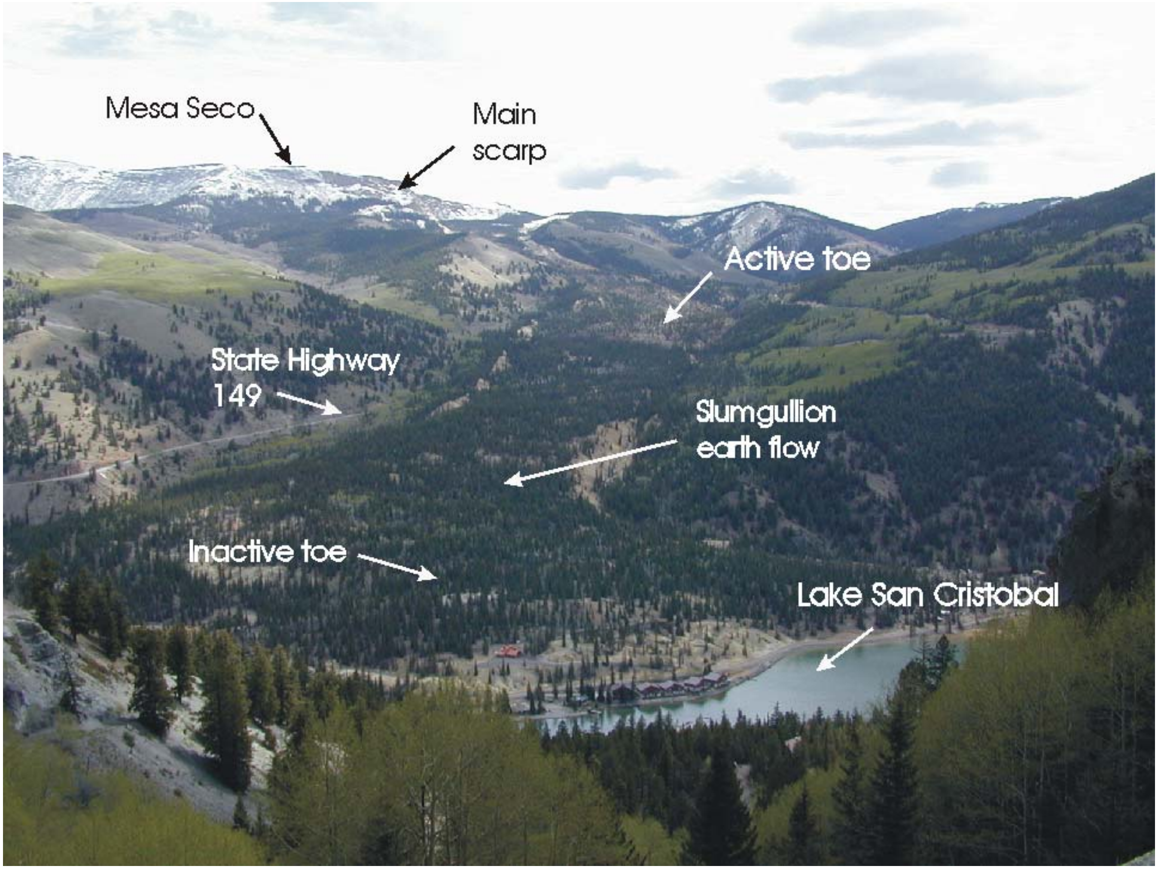

Figure 2. A view of the Slumgullion earth flow. View is to the northeast from southwest of the inactive toe and Lake San Cristobal.

blocked the Lake Fork and caused the impoundment of Lake San Cristobal 700-900 years ago; and a third episode, a collapse of the northwestern part of the main scarp (Figures 1 and 2), occurred about 300 years ago, and initiated the current episode of movement.

CHARACTERISTICS OF THE ACTIVE EARTH FLOW

A mixture of compressional and extensional features occurs at the head of the active earth flow (Figure 3). The extensional features are normal faults and tension cracks while the compressional features are thrust faults and folds. All of these features are related to the collapse of part of the main scarp (Fleming et al., 1999). The earth flow is about 985 ft (300 m) wide at the head. Below the active source area the earth flow bends toward the west, exhibits many extensional features such as normal-fault scarps and tension cracks, and its width decreases to about 750 ft (230 m). The earth-flow neck, where the flow constricts to its minimum width of 490 ft (150 m), is downslope from this area. The flow widens through lateral steps and pull-apart basins downslope from the neck (Fleming et al., 1999). Ponds and pond sediments occur both uphill and downhill from the neck of the earth flow. The location of these ponds has remained fixed while earth-flow material travels downslope through the pond sites.

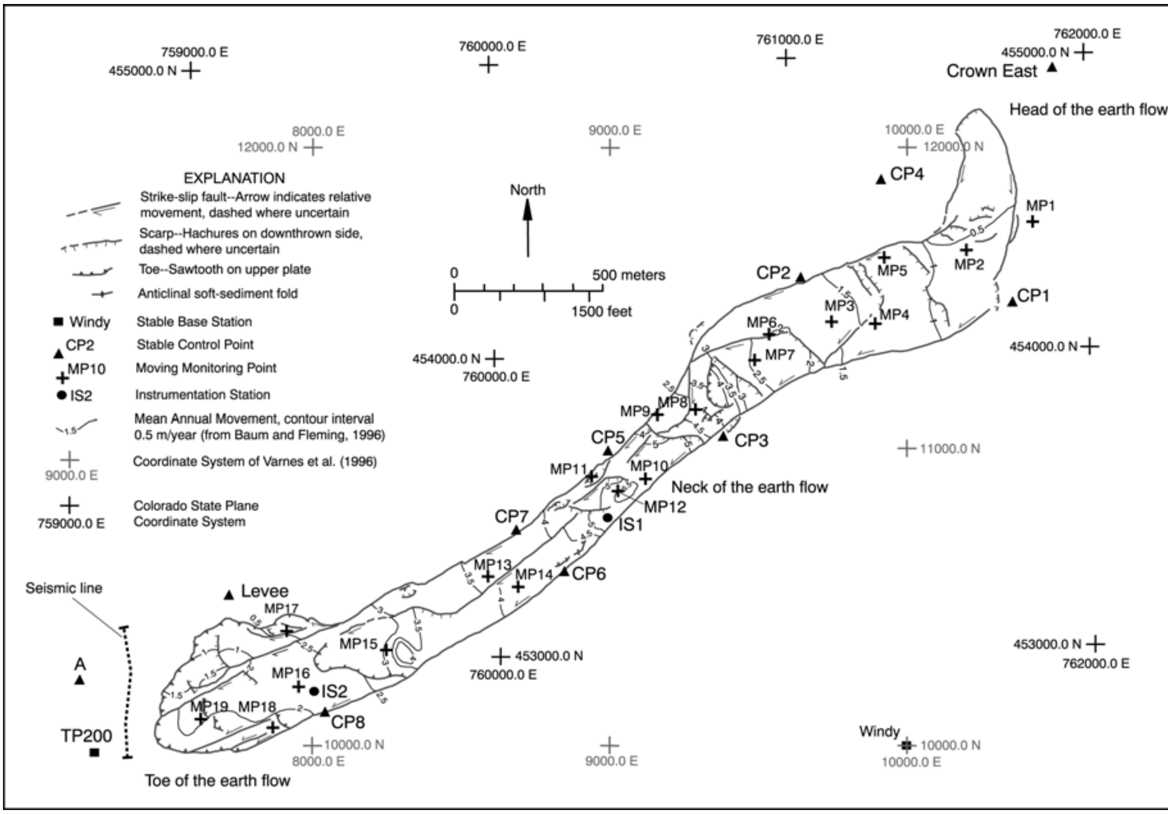

Figure 3. Map of the active part of the Slumgullion earth flow showing structural elements, contours of mean annual displacement, GPS base stations, control points, and monitoring points. Map modified from Baum & Fleming (1996).

The active toe of the earth flow is rounded in plan view, is 1,476 ft (450 m) wide, and has steep slopes (average gradient of 18˚) up to 130 ft (40 m) high. The active toe is advancing across the surface of the old, inactive earth flow, upsetting and burying trees at the base of the earth flow. Compressional features dominate on the earth-flow toe (Fleming et al., 1999).

Levees are a dramatic feature of the Slumgullion earth flow (Figure 4). These features appear to result from shear displacement on strike-slip faults within and along both flanks of the earth flow.

MONITORING OF THE ACTIVE EARTH FLOW

Crandell & Varnes (1961) performed the first investigation of the rates of movement of the Slumgullion earth flow. By measuring changes in staked survey lines from 1958 to 1968 they found average displacement rates of 20 ft/yr (6 m/yr) in the earth-flow neck and average advancement rates of 3 ft/yr (1 m/yr) of the earth-flow toe.

Figure 4. Levees along the south side of the earth flow above control point CP8 (Figure 3). View to the southwest. The inactive levee is to the left. The currently active levee is at the center and is associated with the active strike-slip fault bounding the south side of the earth flow.

Aerial photographs of the earth flow were obtained in 1985 by the Colorado Geological Survey (1:12,000 scale) and in 1990 and 2000 by the USGS (1:6,000 scale). Smith (1996) measured movement of over 300 natural points on the 1985 and 1990 photographs by photogrammetric methods. Total measured displacements from 1985 to 1990 ranged from less than 0.82 ft (0.25 m) to more than 82 ft (25 m), with the greatest displacements occurring in the earth-flow neck. Powers & Chiarle (1996) measured more than 800 horizontal displacement vectors from the 1985 and 1990 aerial photographs using digital, ortho-rectified images. They obtained results consistent with those derived from Smith's (1996) photogrammetric study.

Jackson et al. (1996) were the first to use GPS to measure surface movements on the active earth flow. With a network of seven monitoring points, they found velocities of 0.5 to 0.6 in/day (1.2 to 1.5 cm/day) in the central parts of the active flow during a 4-day period in June 1993. Also in 1993, Gomberg et al. (1995) observed that displacement of earth-flow material occurs along discrete faults that exhibit a combination of brittle failure and stable sliding. Brittle failure was evidenced by "slidequakes" measured with a portable seismic network, and stable sliding events were measured with a buried, digital, high-precision creepmeter.

From July 1998 to March 2002, measurements of earth-flow movement made by GPS surveys of nineteen monitoring points and four extensometers (two at each instrumentation station shown in Figure 3) showed that the flow moved continuously, but that daily velocities varied on a seasonal basis (Coe et al., 2003). Earth-flow velocity increased in response to snowmelt and rainfall, and decreased during dry periods. The time between rainfall, shallow pore pressure response, and earth-flow acceleration was less than several weeks (Figure 5).

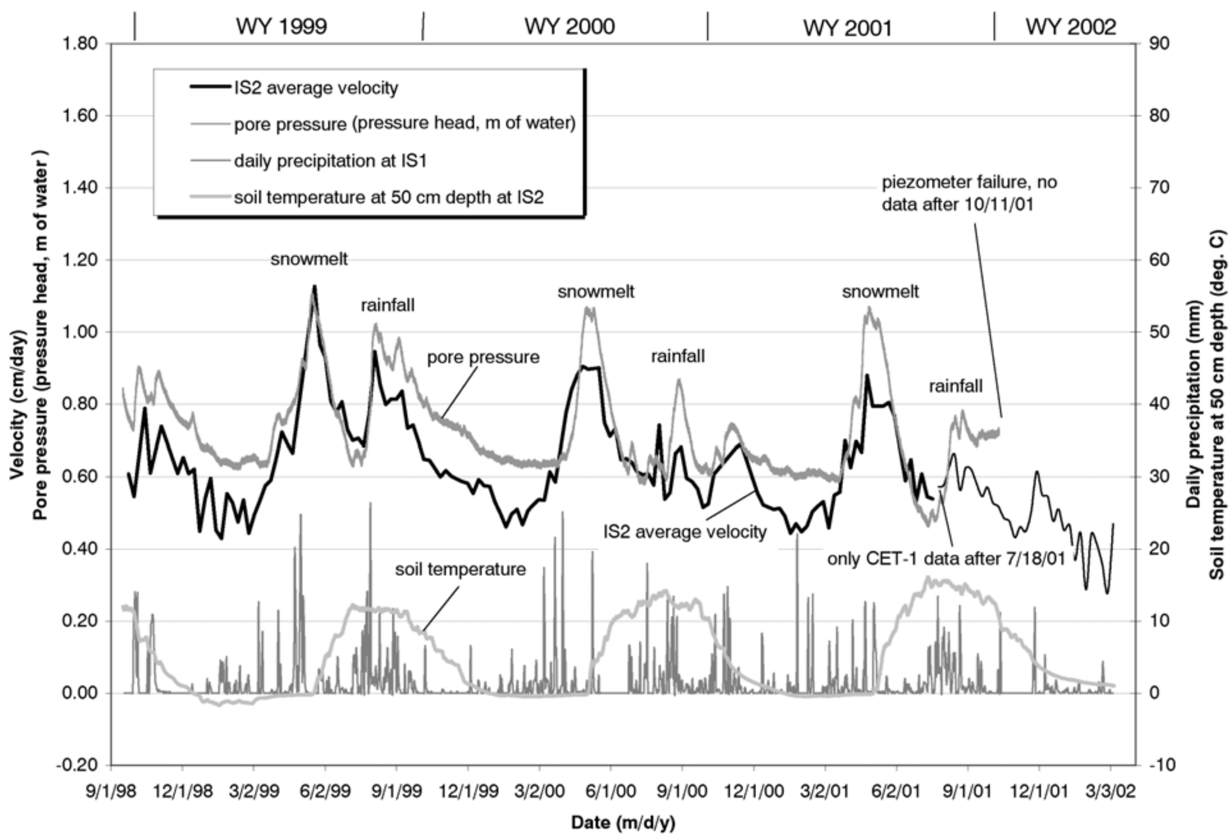

Figure 5. Diagram showing earth-flow velocity, soil temperature, and pore pressure recorded at station IS2 (Figure 3) and daily precipitation recorded at station IS1 (Figure 3). Pore pressure is shown as pressure head in meters of water above the piezometer, which is 6.6 ft (2.0 m) below the landslide surface. Water years (WY), defined as the period between October 1 and September 30, are shown at the top of the diagram. From Coe et al. (2003).

The lowest velocities occurred in mid-winter when air temperatures were at or near annual minimums, the near-surface earth-flow material was frozen, and water was stored on the earth-flow surface as snow. Coe et al. (2003) suggest that variability in velocities is primarily controlled by the availability of surface water from melting snow or rainfall, and that surface water quickly infiltrates the earth flow through patches of bouldery debris or fractures that are created by continuous movement. They also suggest that the continuous, but seasonally variable, movement observed at Slumgullion fits the "bathtub" model for landslide movement. In the bathtub model, described by Baum & Reid (2000), a landslide is isolated both mechanically and hydrologically from adjacent materials by low-permeability clays. These clays cause the landslide to retain water, thus allowing the landslide to respond rapidly to precipitation and snowmelt.

In the summer and fall of 2001, BYU acquired six sets of Synthetic Aperture Radar (SAR) data over the area of the active earth flow. These radar data are currently (April 2003) being interpreted.

The results of monitoring studies to date indicate that annual movements and daily velocities are smallest at the head and toe of the earth flow and largest in the central, narrow neck of the flow (Figure 6). However, movements and velocities deviate from this distribution in areas where they are affected by major structural elements within the earth flow (Baum & Fleming, 1996; Fleming et al., 1999; Coe et al., 2003). Coe et al. (2003) compare annual movements measured between 1998 and 2002 with those measured between 1958 and 1990. Their comparison reveals that, in general, annual movements measured between 1998 and 2002 on the lower and middle parts of the earth flow were greater than any previously documented, whereas annual movements measured between 1998 and 2002 on the upper part of the earth flow were less than previously documented. This implies that the driving forces responsible for moving the upper part of the earth flow may be less than they have been in the past (Coe et al., 2003).

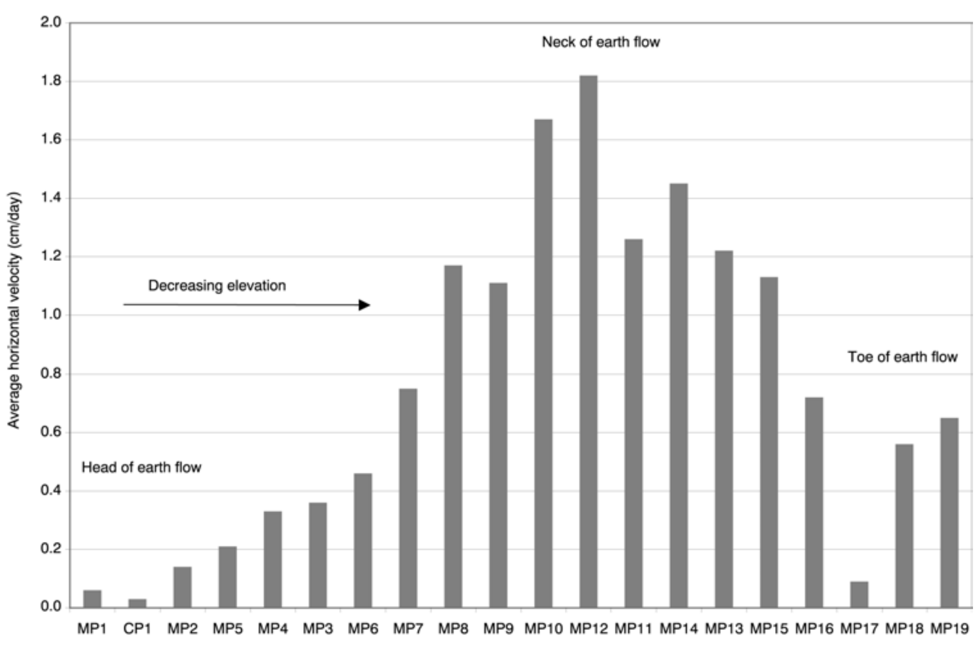

Figure 6. Bar graph showing the average daily horizontal velocity of GPS monitoring points (shown in Figure 3) measured between July 1998 and March 2002. From Coe et al. (2003).

MONITORING OF THE INACTIVE EARTH FLOW

The upper, active part of the earth flow overrides the older, inactive part along the bulging and raveling active toe. Loading exerted by the moving toe causes deformation of the inactive deposits in front of the toe, as shown by leveling surveys of this part of the inactive earth flow. To date, these surveys, which began in 1991 (Varnes et al., 1996), indicate that the inactive part of the earth flow is being depressed in the range 0.2-0.8 in/yr (5-20 mm/yr) by the load of the advancing toe. This depression decreases away from the active front. However, some points close to the toe have significantly risen (0.8-3.2 in (20-80 mm) in 2 years, from 1991 to 1993). These deformations are related to elastic and plastic responses of the inactive deposits to loading by the active toe (Savage et al., 1996).

The inactive part of the earth flow is crossed by Colorado State Highway 149 about 250 m downslope from the active toe. Currently, State Highway 149 appears to be unaffected by deformation below the active toe.

CONCLUSIONS

This review summarizes the long and diverse record of investigations performed at the Slumgullion earth flow. Nearly constant movement makes Slumgullion an excellent, large-scale natural laboratory for such investigations. The active part of this earth flow repeatedly creates and destroys surface features, providing a dynamic record of earth-flow deformation. Displacement data collected during the past 40 years by surveying, photogrammetric, field-instrumentation, and GPS methods reveal that surface velocities vary seasonally and, in general, are inversely proportional to the width of the active earth flow. Maximum surface velocities of 20-23 ft/yr (6-7 m/yr) occur in the narrowest region of the earth flow, and surface velocities of 3-7 ft/yr (1-2 m/yr) occur in the head and toe regions. Leveling surveys indicate that the inactive part of the Slumgullion earth flow is responding to changing loads caused by the advancing toe with a general depression of the ground in front of the advancing toe.

Recent work has shown that earth-flow velocity is closely correlated in time with fluctuations in the hydrologic input at the earth-flow surface. These observations imply that pore pressures at the basal surface(s) respond quickly to moisture flux at the ground surface. The surface movement of the earth flow has been well studied and is currently being monitored, but little is known about the mechanics, materials, and hydrology of the earth flow at depth. To date (April, 2003) there have been no drill holes that have penetrated the earth flow to any appreciable depth. The remote location and rugged terrain of the earth flow present significant difficulties for any proposed drilling program. However, information from such a drilling program would provide a framework for future research questions that address: 1) the depth, geometry, and materials of the basal surface(s) of the currently active earth flow and the older, inactive flows; 2) the role of climate variability and its influence on pore-pressure fluctuation and earth-flow movement rate; and 3) evolution of the earth flow over its history.

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DAMS BUILT ON PRE-EXISTING LANDSLIDES IN COLORADO

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Key terms: landslide, rock slide, rock fall, debris flow, earth flow, dam, abutment

ABSTRACT

In the first half of the 20th century, the geologic profession was in its infancy in Colorado, and dams often were built where landslides provided valley constrictions, often without expert site examination. Only the most important projects were subjected to careful geologic examination. Thus, dams were often built without complete understanding of the possible geotechnical problems involved in foundations or abutments. Most of these dams are still in existence, although many have undergone costly repairs because of leakage or stability problems. Today, however, every effort is made in the selection of dam sites to provide foundations and abutments that are generally impervious and capable of withstanding the stresses imposed by the proposed dam and reservoir. Any pre-existing landslide in the "footprint" of a proposed dam is carefully investigated. If a landslide is recognized at a proposed damsite, the landslide deposits commonly are avoided in siting or are removed during stripping of the dam foundation and abutment contacts. However, it is often found to be technically feasible and economically desirable to site and construct dams on known landslides or their remnants. In these cases, proven preventive and/or remedial measures have been used to ensure the stability of the foundations and abutments, and to reduce seepage to acceptable levels.

By literature search, technical interviews, and field inventory, I have located 56 *large* (i.e., at least 33-ft (10-m) high) dams in Colorado that have been built on pre-existing landslides. All 56 of these dams are located in the western two thirds of the state, i.e., in the Rocky Mountains. Of the 56 dams, 52 are earthfill, one is earthfill/rockfill, and two are rockfill; these are flexible dam types that are better-suited to satisfactory performance on the possibly unstable foundations provided by landslides than are more-rigid concrete dams. The 56 dams were built on a variety of landslide types, ranging from Taylor Dam, the right abutment of which consists of rock-fall talus, to Platoro Dam, whose left abutment is a large andesite slump block, to Gross Dam (the only concrete dam in the study), which is on an ancient gravity slip in granite in a steep-walled canyon. However, most of the sites are related to slides/slumps in soft sedimentary rocks, such as the Mancos Shale. In general, dams built on pre-existing landslides in Colorado have performed reasonably well, although some have been subject to foundation or abutment seepage problems that have required substantial maintenance expenditures. This paper includes a table that summarizes landslide conditions and remedial measures at each of the 56 dams.

INTRODUCTION

Background

> "Many major slides have in the past blocked up river valleys, the resulting constrictions giving dam sites which appear at first sight to be admirable. In view of the nature of the material in slides, and its unconsolidated and often disturbed conditions, sites of this kind may often prove to be far from ideal. Dams have, however, been successfully founded at such sites." (Legget, 1939, pp. 225-226).

In the early part of the 20^{th} century, a few experts recognized and noted the problems posed by pre-existing landslides in foundations or abutments of newly built and proposed dams. In Colorado, such problems were first pointed out by W.W. Atwood (1918) in his pioneering U.S. Geological Survey Bulletin 685, *Relation of Landslides and Glacial Deposits to Reservoir Sites in the San Juan Mountains, Colorado*. At that time, the geologic profession was in its infancy in Colorado and numerous dams were built at sites where landslides provided valley constrictions, often without expert examination. Only the most important projects were subject to careful geologic examination. Thus, dams were often built without complete understanding of the possible geotechnical problems occurring in foundations or abutments. Most of these dams are still in existence, although many have undergone costly repairs because of leakage or stability problems.

Today, however, in the selection of a damsite every effort is usually made to provide a foundation and abutments that are relatively impervious and capable of withstanding the stresses imposed by the proposed dam and reservoir under all probable loading conditions. Thus, permeability and stability must be considered during the selection, site preparation, construction, and operation of the structure (Záruba, 1979; Weaver, 1989).

As noted by Weaver (1989), "Landslides, if recognized prior to construction, presumably are avoided or are removed during stripping of the dam foundation. However, landslides of significant size sometimes occur during the course of stripping operations for a dam on weak rock foundations, and must be left in place." It has often been found to be technically feasible and economically desirable to site and construct dams on known landslides or on their remains after most of the material has been stripped from the site. The stability of the landslide or its remnant can be enhanced by the buttressing effect of the dam itself, often augmented by berms acting as buttresses, by anchors, or by retaining structures. In addition, problems caused by seepage through the landslide deposits usually can be remedied by means of surface and/or subsurface drains, impervious cutoffs or membranes, grout curtains, or other preventive or remedial measures.

Presentation of Data

Table 1 presents 56 cases noted in a survey of "large" dams built on pre-existing landslides in Colorado or on the remnants of partly excavated landslides, or on construction-caused landslides, part or all of which were left in place as part of foundations or abutments. This tabulation does not include dams that have been built on sites from which landslide materials have been entirely removed from the dam footprint before dam construction. For cases in which " the dam had specially difficult foundation conditions," the International Commission on Large Dams (ICOLD) uses a minimum height of 10 m (33 ft) as the standard to define a "large dam" (International Commission on Large Dams, 1977). Because dams built on landslides appear to fit this definition, I have accepted this height as the minimum in defining large dams for this study. The term "landslide" will be used to include all types of gravitational mass movements, i.e., falls, slides, slumps, avalanches, etc.

Each case includes dam and stream names, location, dam type and purpose, year of completion and type of owner, dam and reservoir dimensions, landslide type and position relative to the dam, comments regarding the landslide and remedial measures, and references and other information sources. Later in this paper the information presented in Table 1 will be summarized and conclusions will be drawn. The object of this tabulation is to provide observations and conclusions to aid in making future siting decisions in cases in which landslides are involved in dam planning.

Sources of Information

The data obtained in Table 1 have been obtained from:

- Published technical papers and reports that have appeared in the geological, engineering, and geotechnical literature.
- Reports issued by the U.S. Bureau of Reclamation.
- State and Federal dam-safety officials, and especially engineers and scientists of the Colorado Division of Water Resources.
- Reports by and interviews with geologic/geotechnical consultants and colleagues experienced in dealing with Colorado landslides and dams.
- Personal experience of the author, including visits to nearly all of the dams listed.

Most physical data on the tabulated dams were obtained from the CD-ROM, *National Inventory of Dams 1996* (Federal Emergency Management Agency, 1996). This inventory of some 76,000 U.S. dams includes 1606 dams in Colorado, 664 of which are at least 33 ft (10 m) high.

Summary of Data

As shown in Table 1, this study has noted 56 dams in Colorado that have been built on pre-existing landslides, or about 8 percent of the 664 Colorado dams that are higher than 33 ft (10 m). All 56 of these dams are located in the western two thirds of the state, i.e., in the Rocky Mountains (Figure 1). Of the 56 dams, 52 are earthfill, one is earthfill/rockfill, and two are rockfill; these are flexible dam types that are better-suited to the possibly unstable foundations provided by landslides than are more-rigid concrete dams.

The *primary* purposes of these dams are: irrigation – 42; recreation – 7; water supply – 6; hydroelectric power – 1. Many of these 56 dams also serve secondary purposes in these same categories, as well as other functions, such as flood control, fish-and-wildlife habitat, fire protection, stock watering, etc.

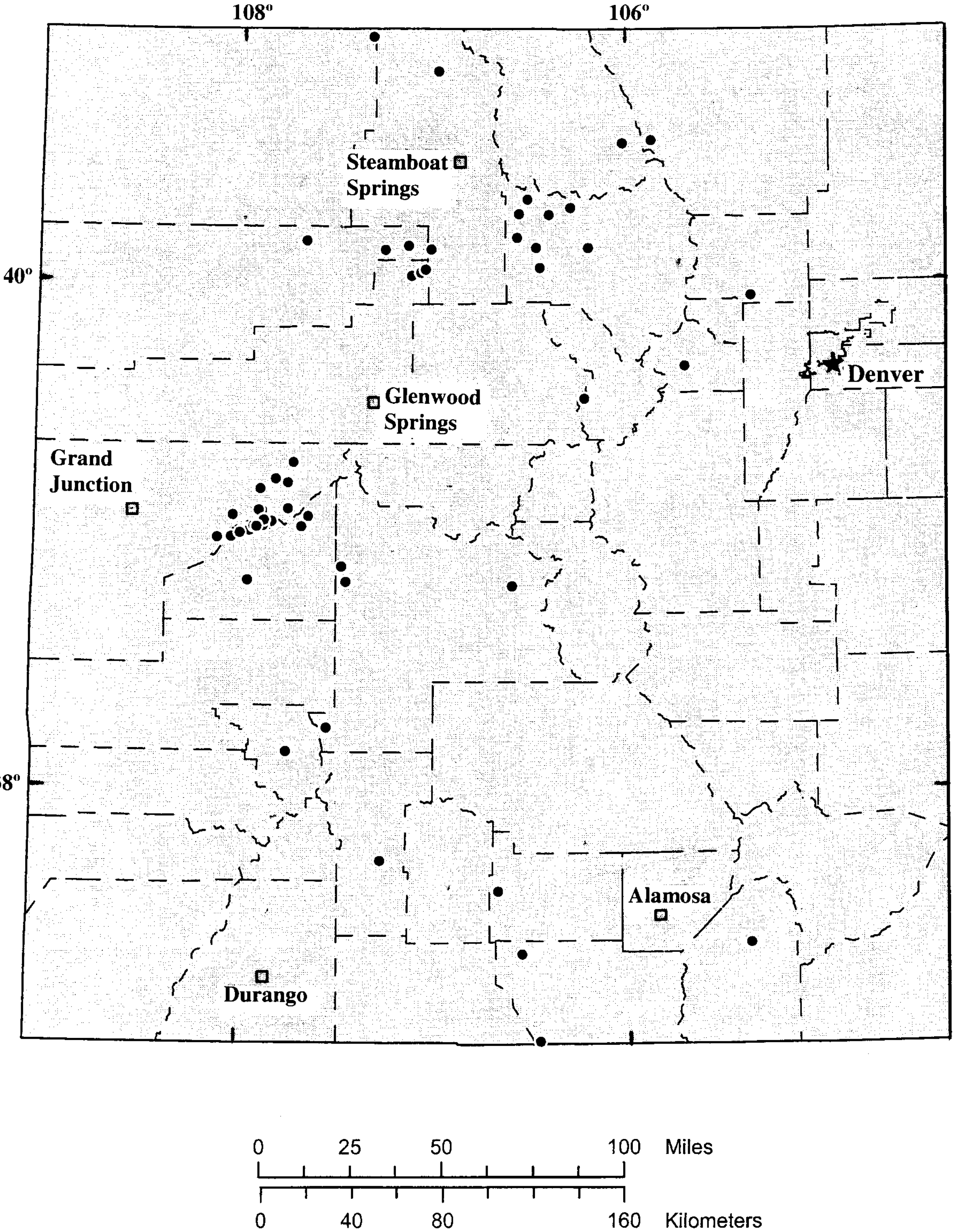

Figure 1. Distribution of 56 dams (solid circles) built on pre-existing landslides throughout the western two thirds of Colorado.

The structural heights of the 56 dams range from a minimum of 33 ft (10 m) to a maximum of 340 ft (104 m). The two highest dams in the study are 340-ft (104-m) high concrete-gravity Gross Dam in Boulder County and 330-ft (101-m) high earthfill Ridgway Dam in Ouray County.

Most of the dams studied (44 of the 56) are less than 100 ft (30 m) high, mostly being locally-owned irrigation dams.

It is not easy to categorize the 56 tabulated cases by landslide type because many of the landslides are "complex," i.e., more than one type of landslide occurred. For each case, I arrived at a "best estimate" of landslide type based on a simplified approximation of the "Varnes landslide classification" (Varnes, 1978; Cruden & Varnes, 1996). In the case of complex landslides, I have attempted to note the primary type of movement. A high percentage of the cases involved slides or slumps in soft rocks, generally shales, siltstones, and soft sandstones, often overlain by indurated volcanic rocks, commonly basalt or andesite. In a few cases, the dams were built on debris-flow, earthflow, or rock-fall (talus) deposits.

IMPORTANT CASE HISTORIES BY LANDSLIDE TYPE

Dams Built on Rock-fall (Including Talus) and Rock-avalanche Deposits

Although other Colorado dams include rock-fall deposits in one abutment or the other, 206-ft (63-m) high Taylor Park Dam on the Taylor River in Gunnison County, a federal earthfill irrigation dam completed in 1937, is noteworthy for its talus-covered right abutment (Figure 2). During planning, geologists and engineers recognized that this abutment was covered by a thick talus cone derived from overlying cliffs of Paleozoic sedimentary rocks. The talus mass consisted predominantly of boulder- to cobble-size sandstone fragments with some fine to coarse gravel and sand. During construction, the talus was removed from under the center and upstream parts of the dam, but was left in place in the area extending from ~75 ft (~23 m) downstream from the centerline to the downstream toe of the dam. The remaining talus in the downstream part of the right abutment poses a very minimal threat of piping; thus far, in spite of some seepage, there has been no evidence of piping. The dam has had a long history of satisfactory performance. However, rock-fall activity from the cliffs above both abutments results in a continuing minor maintenance problem; scaling was performed at the downstream right abutment in 1994.

Originally constructed in 1905, Clear Lake Dam, a 40-ft (12-m) high rockfill dam, one of a series of hydroelectric dams on South Clear Creek just south of the town of Georgetown in the Colorado Front Range, is founded on talus and rock-slide material (Widmann and Miersemann, 2001; Hammer, 2002) (Figure 3). Because of the talus, the foundation and abutments are very pervious. As noted by Hammer (2002): "In 1956 plans were filed to correct seepage concerns that were described as being of a magnitude that matched reservoir inflow." In 1997, seepage and piping occurred through left abutment talus, resulting in "sinkholes" (Hammer (2002)). Grouting was performed in 2002 to reduce seepage.

In an incident not directly related to the topic of this paper (i.e., no landslide in the dam "footprint"), in 1965 three workmen were killed by a slide triggered by construction of a diversion tunnel for carrying water around under-construction Lower Cabin Creek Dam (Figure 3), another hydroelectric dam located on South Clear Creek 0.7 mi (1.1 km) upstream from Clear Lake Dam (Myers, 1965). Although the valley bottom at the site of 95-ft (29-m) high, earthfill

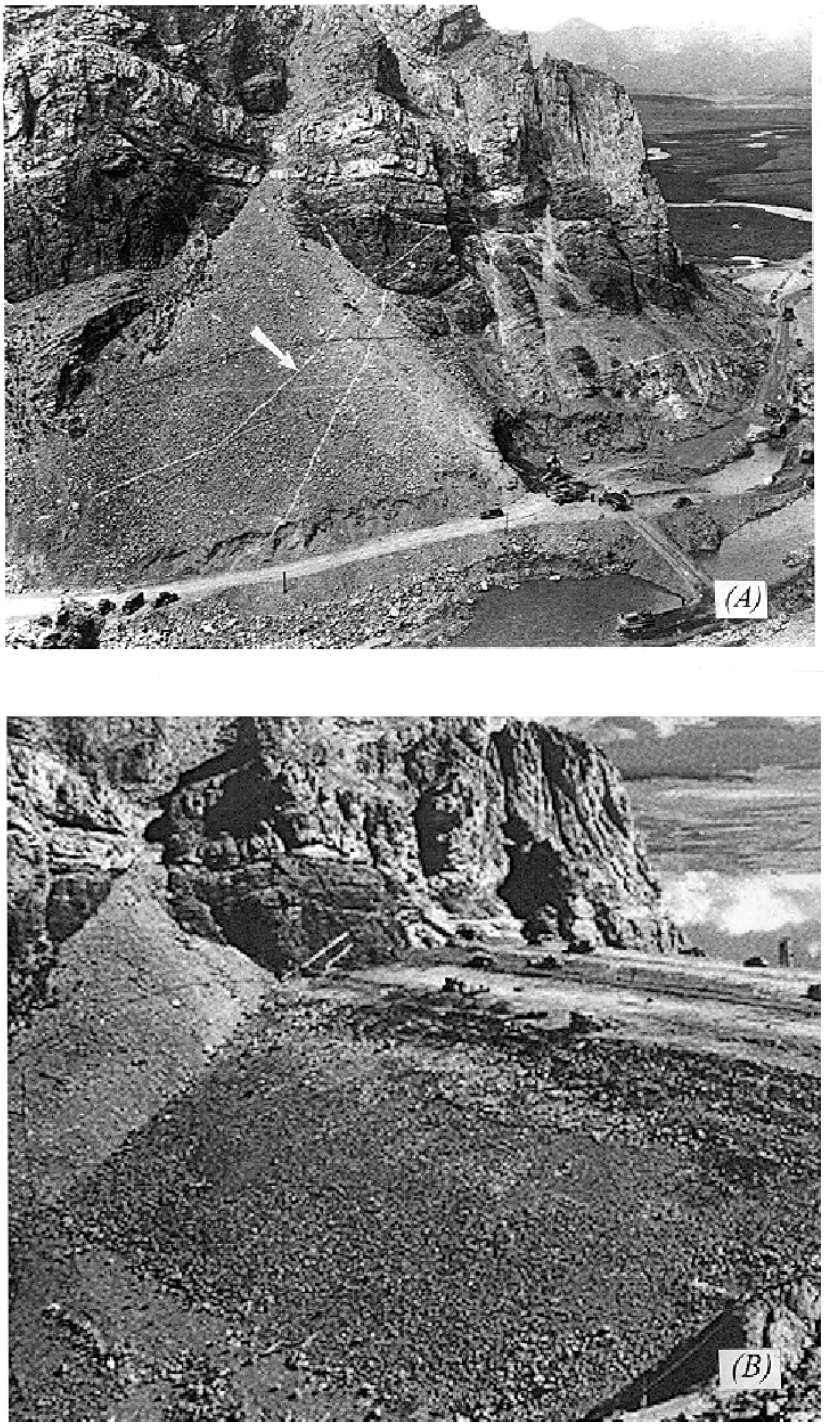

Figure 2. Views before (*A)* and after (*B)* construction of Taylor Park Dam, central Colorado. Note talus deposit that serves as the right abutment of the dam, and the chalk line (white arrow) at the left center of photograph (*A)* showing the location of the right end of the future dam on the talus cone. (1937 photographs by A.A. Whitmore, U.S. Bureau of Reclamation)

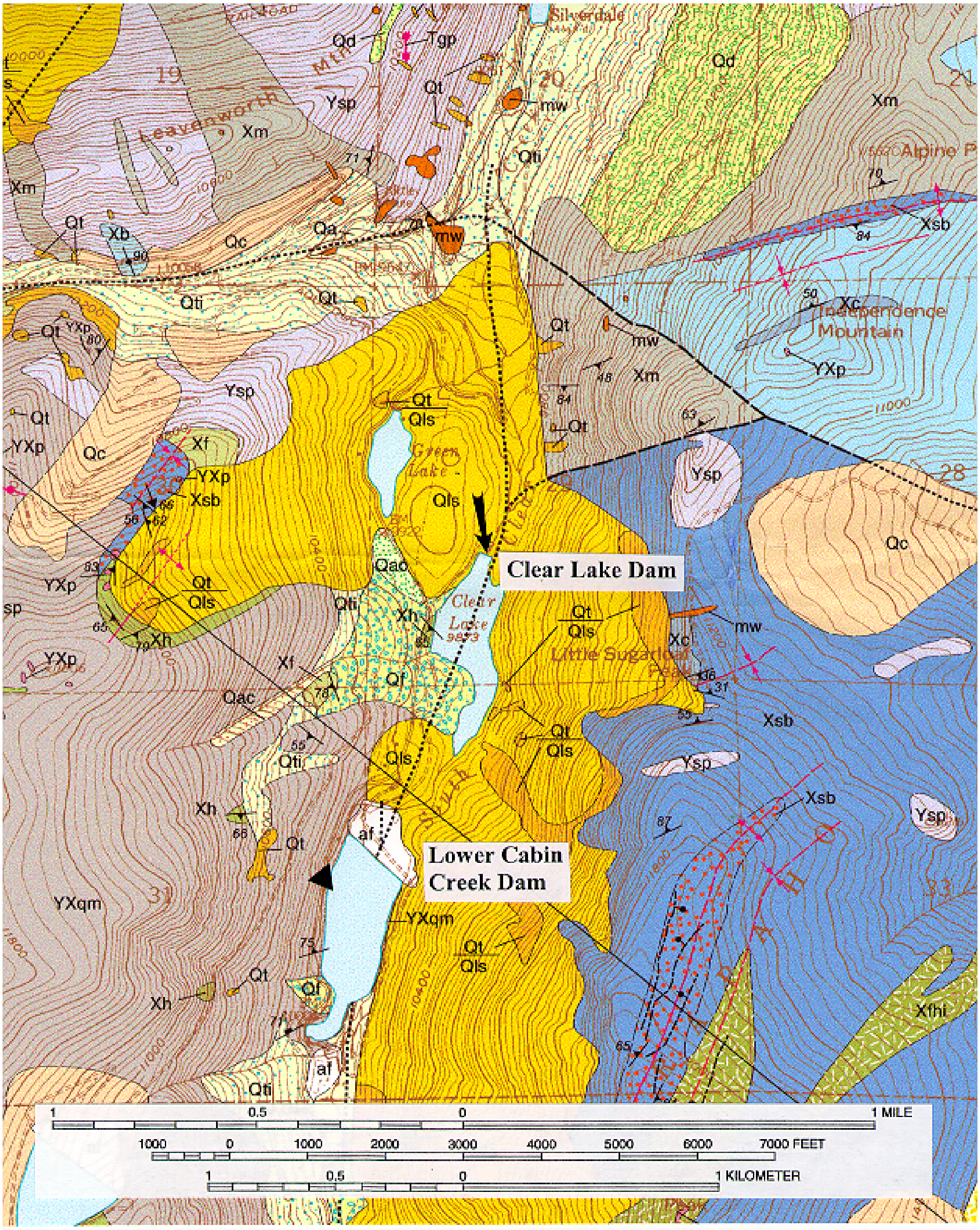

Figure 3. Surficial-geologic map showing location of Clear Lake Dam (arrow) on rock-slide deposits (Qls) and talus (Hammer (2002) along Clear Creek 2 mi (3 km) south of the town of Georgetown in the Colorado Front Range (after Widmann & Miersemann, 2001). Also shown is the location of Lower Cabin Creek Dam, the foundation of which reportedly was excavated through landslide deposits to bedrock

Lower Cabin Creek Dam, as well as the entire right valley wall above Lower Cabin Creek Reservoir, has been mapped by Widmann and Miersemann (2001) as “Landslide deposits” (Figure 3), landslide material in the footprint of the dam reportedly was removed during construction and the dam is founded on bedrock. Thus, Lower Cabin Creek Dam has not been included in Table 1.

Georgetown Lake (Harry Locke) Dam is located on Clear Creek, 4.5 mi (7.2 km) downstream (i.e., north) from Clear Lake Dam. This 30-ft (9-m) high earthfill dam, built for recreation and hydroelectric power, is too small to be included in the Table 1 compilation of “large” dams. However, it is worthy of mention because it is an outstanding example of a dam that has been built in contact with the toe of a large rock avalanche, and because it is easily viewed from Interstate 70 at a point 2 mi (3.2 km) north (i.e., toward Denver) from downtown Georgetown (Figure 4). The original dam at this site was built in 1906; this structure failed in June 1956 due to overtopping during flood stage (Woodward-Clyde & Associates, 1970). There is a strong possibility that the inherent weakness of the landslide material that comprised the right abutment of the dam contributed to the failure. The current dam, constructed in 1960, is performing satisfactorily.

Figure 4. Georgetown Lake (Harry Locke) Dam (lower left in photo) and large rock avalanche that forms its right abutment (center of photo). View is from left valley wall above Interstate 70, 2 mi (3.2 km) north (i.e., toward Denver) of downtown Georgetown. (2000 photo)

Hogchute Dam, a 56-ft (17-m) high municipal earthfill water-supply dam on the southwest side of the Grand Mesa in western Colorado also has considerable basalt rock-fall talus (from the cliffs of the Grand Mesa) in its right abutment. This material has had no negative effect on dam performance.

Dams Built on Massive Rock Slides/Slumps and Glide Blocks

The two areas of Colorado with the largest numbers of dams built on massive landslides/ glide blocks are the landslide area adjoining the Grand Mesa in western Colorado and the mountains of north-central Colorado. In both of these areas, there are very large rock slides and slumps that are generally the result of the failure of weak sedimentary or volcanic rocks overlain by more-resistant volcanics, usually basalt.

Grand Mesa Landslide Area – Rising to an elevation of 10,800 ft, the Grand Mesa is an ~50-mi^2 (~130 km^2) plateau remnant in western Colorado that is capped by continuous, undisturbed late Tertiary basalt flows that slope gently to the west. The basalt flows are underlain by a sequence of claystone, conglomerate, and sandstone, which overlies the Tertiary Green River Formation. These relatively weaker sedimentary rocks have failed, forming steep cliffs, 100-500 ft (30-150-m) high, which surround the upland surface of the Grand Mesa (Yeend, 1969, 1973). A very irregular surface produced by huge slumps and modified by glaciation extends outward from the base of the basalt cliffs (Baum and Odum, 1996, 2003). The slump blocks are tilted back toward the undisturbed part of the mesa, forming long, narrow ridges that parallel the mesa edge. Numerous lakes have been formed as a result of slumping and subsequent glaciation; many of these natural lakes have been increased in depth by the addition of man-made dams. East of today's Grand Mesa, the landslide bench is extensive. Overall, this huge area of slump-blocks extends about 25 mi (70 km) from east to west and 12 mi (20 km) from north to south, most of it lying to the east of the basalt remnant of the Grand Mesa (Figure 5).

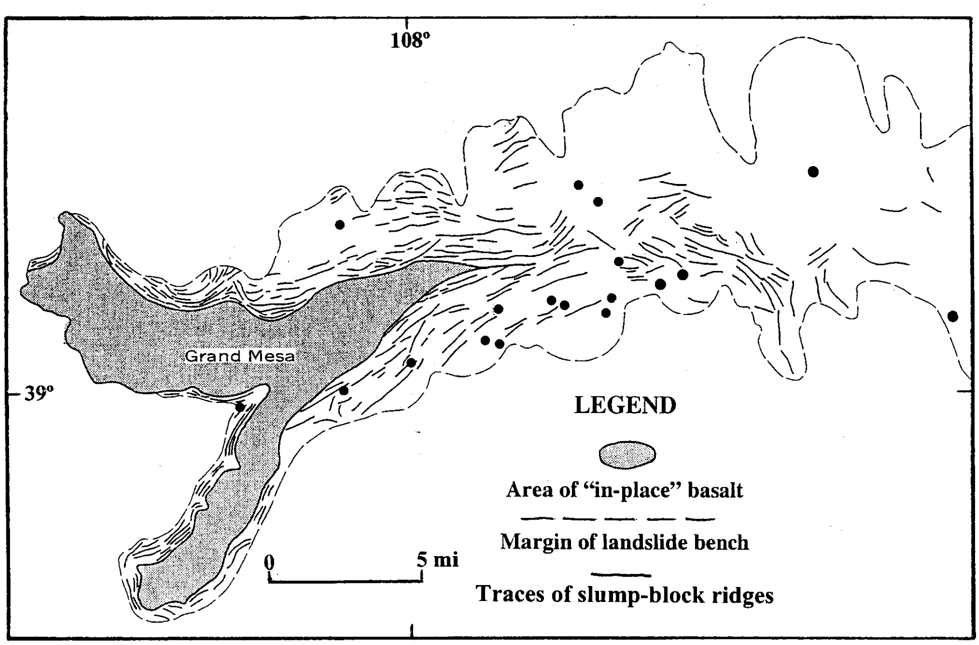

Figure 5. Locations of 18 dams (solid circles) on landslide bench and slump-block landslides derived from basalt plateau of the Grand Mesa, western Colorado. One other dam (Overland Dam) is located on these landslide deposits less than one mile (1.6 km) east of the mapped eastern border shown in this figure. (Landslide map after Yeend (1969))

As became obvious early in the 20th century, the irregular surface topography of this large landslide bench was well-suited for the easy impoundment of snowmelt by dams and reservoirs. Beginning early in the century, many dams and reservoirs were built on these landslides and intermingled glacial deposits. Table 1 lists 19 irrigation, water supply, and/or recreation dams that are founded on Grand Mesa landslides, ranging in structural height from 33 to 85 ft (10 to 26 m)(there are no larger dams on the Grand Mesa). Noted in Table 1 as being on this landslide bench or on rock-fall deposits from Grand Mesa cliffs are the following dams: Atkinson, Big Beaver, Bonham, Cedar Mesa, Eggleston, Goodenough #2, Granby #12, Hogchute, Kehmeier, Kennicott Slough, Kiser Slough, Knox, McKoon, Monument #1, Overland #1, Park, Vela, Ward Creek, and Young's Creek Nos. 1 and 2. Most of these dams have had foundation or abutment seepage problems, often requiring repairs. None have manifested stability problems or pose downstream hazards.

Mountains of North-Central Colorado – North-central Colorado includes several subranges of the Rocky Mountains: generally from west to east, the most important subranges in this area are the Elkhead Mountains, the Flattop Mountains, the Park Range, the Gore Range, and the Colorado Front Range. In these ranges, large rock slides consisting of resistant rocks (usually volcanics) overlying softer rocks (usually shales, siltstones, and sandstones) are common. These rock slides often have occurred upon retreat of Late Pleistocene valley glaciers as massive slides from valley walls. They have narrowed high mountain valleys, forming opportune sites for the location of dams. Examples have been provided by Stillwater #1, Upper Stillwater, Yamcolo, Poose Creek, and Sheriff Dams, irrigation and recreation dams located in valleys that drain from the northeast slopes of the Flattop Mountains, which consist of Tertiary basalts overlying shales and sandstones. A similar example is provided by Joe Wright Dam, a large water-supply dam on eastward-flowing Joe Wright Creek in the Colorado Front Range. In spite of these dams being located partially or entirely on the toes of landslides, the slides have had no negative effects on dam performance; this may be at least partly due to the massive bulk of the slides, which aids stability and inhibits seepage.

Three other dams and their reservoirs in north-central Colorado are located entirely on glide blocks in soft rocks. Jones #2 Dam, a 40-ft (12-m) high earthfill dam built in 1887, which serves as the main water-supply dam for the town of Kremmling, lies on a massive, prehistoric 5-mi^2 (16-km^2) glide block in Niobrara and Dakota shales and sandstones, where Middle Park abuts the Gore Range. D.D.& E. Wise (Aldrich Lake) Dam, a 41-ft (12-m) high earthfill irrigation dam lies on the eastern edge of a massive 5-mi^2 (16-km^2) landslide in Mancos Shale in the Danforth Hills north of the White River. Lower Cogdill Dam and Reservoir lie entirely on an ~7-mi^2 (~18 km^2) glide block overlying Lewis Shale and Mesaverde Group sandstones and shales on the west slope of the Elk Head Mountains in northwestern Colorado. Minor seepage issues from toe drains at Jones #2 Dam. Lower Cogdill and D.D.& E. Wise Dams have shown no distress resulting from their landslide foundations and abutments.

Matheson Dam, a 60-ft (18-m) high irrigation dam, was built on the northeast edge of an ~7-mi^2 (~18 km^2) landslide mass (Tertiary volcanics overlying Morrison Fm. shales and sandstones) in the Rabbit Ears Range northeast of Kremmling. There has been some seepage through the landslide right abutment of this dam, probably related to the landslide materials.

Dams Built on Individual Rock Slides/Slumps

Numerous Colorado dams have been built on individual rock slides/slumps, mostly in cases where the original slope failure occurred in resistant volcanic rocks overlying weaker sedimentary or volcanic rocks. Four of the most noteworthy of these, Rio Grande, Platoro, Silver Jack, and Ridgway Dams, are large irrigation dams located in the San Juan Mountains and vicinity in southwestern Colorado.

Rio Grande Dam – Rio Grande Dam (originally named Farmers Union Dam, Figure 6) was constructed in 1916 on the headwaters of the Rio Grande River as a 116-ft (35-m) high earthfill, irrigation dam. It was noted by Atwood (1918) that the left end of the dam abuts the toe of a large rock slide in Tertiary andesite and underlying tuff. Seepage through this abutment led to rebuilding the right end of the dam in the 1990's, including installation of horizontal and French drains, a retaining wall, and piezometers and inclinometers.

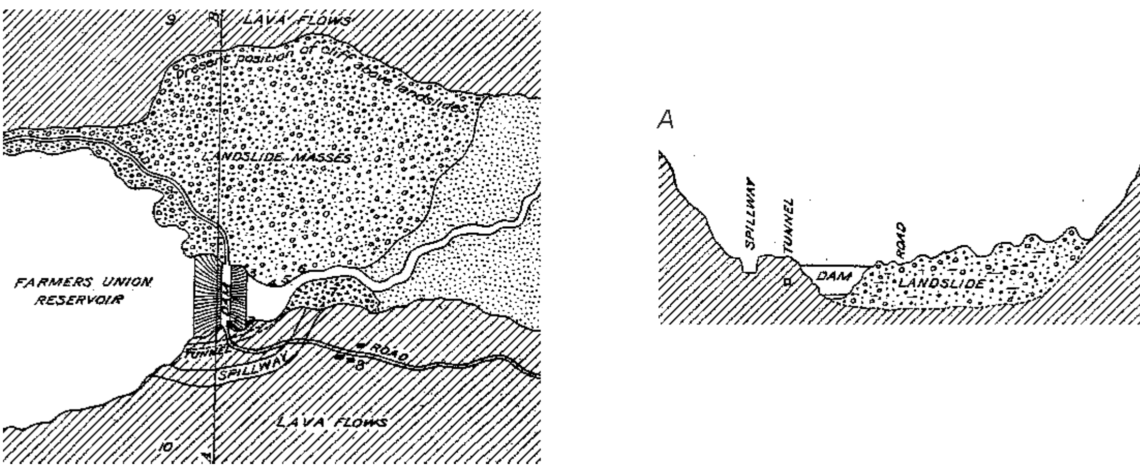

Figure 6. Original 1918 sketch map (a) and cross section (b) along line *A-B*, showing geologic conditions at Farmers Union (now Rio Grande) Dam on the headwaters of the Rio Grande River, southwestern Colorado (Atwood, 1918).

Platoro Dam – Completed in 1951, Platoro Dam is a 165-ft (50-m) high federal rockfill irrigation dam on the Conejos River in the southern San Juan Mountains. The left abutment of the dam is the toe of a large (>0.5-mi (>0.8-km) long) andesite slump block, the existence of which was known before construction. There have been no stability or seepage problems.

Silver Jack Dam – Silver Jack Dam is a 173-ft (53-m) high federal earthfill irrigation dam on Cimarron Creek, a northward-flowing tributary of the Gunnison River that originates on the north flank of the San Juan Mountains. The right end of the dam abuts the toe of a massive landslide in glacial deposits and Tertiary volcanic rocks (tuff-agglomerate complex) that overlie Mancos Shale along the east valley wall of Cimarron Creek. During construction, excavation for the right abutment and spillway of the dam caused a 500,000-yd^3 (380,000-m^3) reactivation of the toe of this landslide, extensively damaging the partially constructed spillway conduit and stilling basin (Logan and Davis, 1973). Steps taken during construction to stabilize and monitor the slide area included the following:

- Relocation and redesign of the spillway chute and chute stilling basin.
- Construction of a 155,300-yd^3 (118,800-m^3) toe-buttress embankment.
- Installation of surface and subsurface (35 horizontal drains) drain systems.
- Relocation of the access road, which previously crossed the slide area.
- Drilling five additional water-observation holes in the slide area to monitor groundwater conditions during reservoir filling.

No movement has occurred in the landslide mass since these measures were installed.

Ridgway Dam – Completed in 1987, Ridgway Dam is a 330-ft (101-m) high federal earthfill irrigation/water supply/recreation dam on the Uncompahgre River in the San Juan Mountains. During construction, reactivation of ancient slides occurred in left-abutment Dakota Group and Morrison Fm. sandstones, siltstones, and mudstones (Von Thun, 1987). More than 40,000 yd^3 (30,000 m^3) of slide debris was removed. In addition, incipient movement of a large slide block was remedied by installation of rock-anchor tendons and tunnel drains in the slide mass. Monitoring instrumentation also was installed. The abutment currently is stable and seepage-free.

Three other major dams have been constructed on rock slides outside the areas noted above – Mountain Home, Fruitgrowers, and Gross Dams:

Mountain Home Dam – Mountain Home Dam is a 153-ft (47-m) high, privately owned earthfill dam, constructed in 1906 primarily for irrigation, but used today also for recreation, as well as fish-and-wildlife habitat. It is located on Trinchera Creek, a tributary of the Rio Grande River, where the San Luis Valley meets the western foothills of the southern Sangre de Cristo Mountains in south-central Colorado. The upper left abutment of Mountain Home Dam is a Pleistocene translational block slide in Pliocene-Miocene basalt underlain by siltstone and sandstone of the Tertiary Santa Fe Formation. This block slide underwent no movement during construction. However, because the overflow spillway crosses the slide block, the possibility of future instability was conjectured; analysis in 1993 indicated no instability problems.

Fruitgrowers Dam – The present 55-ft (17-m) high Fruitgrowers Dam (Figure 7) on Alfalfa Run, a tributary of the Gunnison River south of the Grand Mesa, was built in 1939 on the site of a 19th-century dam that had failed in 1937 due to faulty design and materials. The left abutment and part of the left foundation of this federal earthfill irrigation dam is an ancient block-glide landslide in Mancos Shale, which has reactivated since the dam was put into service. Slow reactivation along the gently dipping failure surface of this block glide presumably tilted the original spillway, which, as a result, was relocated to the right end of the dam in 1987. As determined by inclinometers, the rate of movement of the left-abutment landslide toward the dam from 1981 to 1999 ranged from 1/3 to one inch (0.8 to 2.5 cm) per year; this movement continues to be monitored.

Gross Dam – Completed in 1955, 340-ft (104-m) high Gross Dam on South Boulder Creek in the Colorado Front Range is the only concrete-gravity dam in this study. As noted by Wahlstrom (1974, pp. 74), deep-seated "gravity-slip surfaces in steep-walled canyons in massive crystalline

Figure 7. Fruitgrowers Dam, a federal irrigation dam in southwestern Colorado, showing landslide topography in Mancos Shale on slope at the left abutment. (1999 photograph)

rock" [granite] were found in both abutments at this water-supply dam for the City of Boulder (Figure 8). Although these "gravity faults" were closely watched during construction, they pose no hazard to the completed dam, which buttresses any possible future movement. Thus, even though this dam is a relatively inflexible concrete structure, this type of slide was primarily a "textbook" case that posed no danger to the dam; it is the only one of its kind noted in Colorado.

Dams Built on Debris-/Earth-flow Deposits

Only four Colorado dams have been built partially on pre-existing debris flows or earth flows: Black Lake #1 and Vega Dams on ancient debris flows and McElroy and North Michigan Creek on earth flows. Of these, Vega Dam deserves special mention.

Vega Dam – Completed in 1959, 162-ft (49-m) high Vega Dam is a federal-government irrigation and recreation dam on Plateau Creek northeast of the Grand Mesa. As mapped by Soule (1988), the dam's left abutment, the upstream part of the right abutment, and nearly all of the reservoir are situated on "Eroded and Man-Made Remains of a Massive Debris Flow Deposit that resulted from a large-scale debris flow that originated on Grand Mesa, approximately 5 mi [8 km] south of the mapped area" (Soule, 1986). The downstream part of the right abutment is on a slide in Wasatch Fm. claystone, mudstone, and sandstone that Soule (1988) has mapped as: "Ancient landslides – These are areas where the landsliding process took place long enough ago that erosion and other surficial processes have considerably modified the form of the deposit." Post-construction slides occurred at both downstream abutment areas. During the late 1980's, a slide encroached on the spillway wall at the right abutment. There appears to be no current activity.

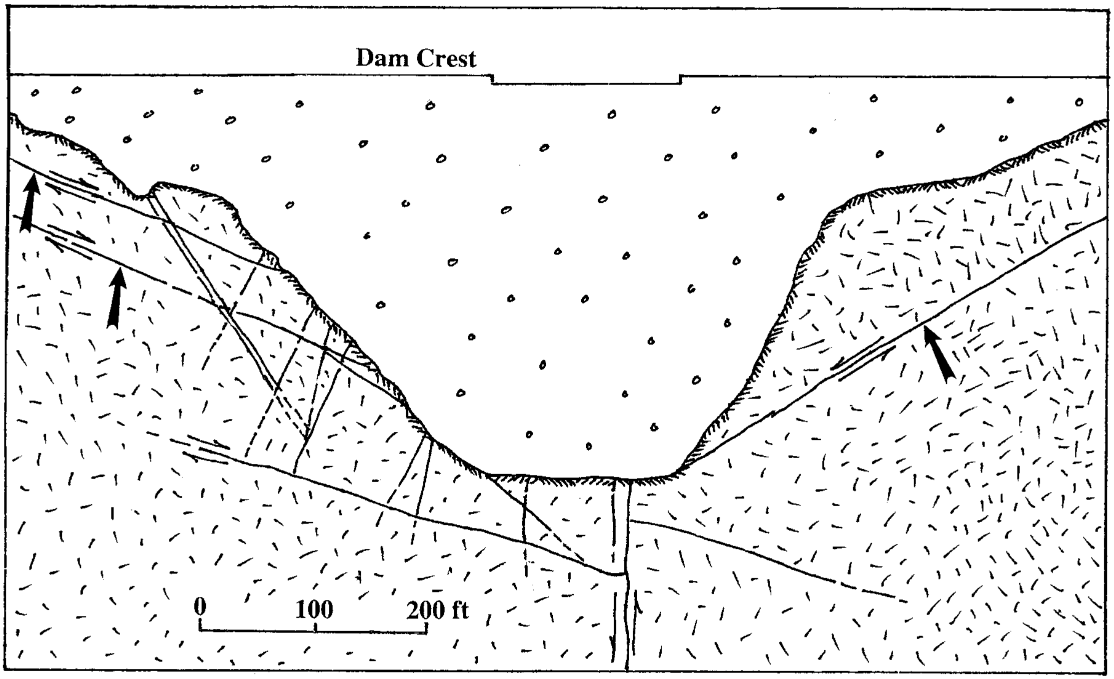

Figure 8. Gravity-slip surfaces (solid arrows) in granitic foundation rocks at Gross Dam, Colorado Front Range (after Wahlstrom, 1974). Faults of tectonic origin were present in the granite before the gravity-slip "faults" developed, and are geologically much older.

PROBLEMS ENCOUNTERED DUE TO CONSTRUCTION OF COLORADO DAMS ON LANDSLIDES

Abutment and Foundation Instability

Of the total of 56 Colorado dams constructed on landslides, 11 encountered abutment or foundation slope failures either during construction or after dam completion. Cases of major slope failures during construction occurred at Silver Jack and Ridgway Dams, which are large federal irrigation dams in the San Juan Mountains, where pre-existing rock slides were reactivated by the construction process. Most of the slide material was removed; however, some was left in place. At Silver Jack Dam, a large slide was stabilized by construction of a 115,300-yd^3 (118,800 m^3) earthfill buttress, installation of horizontal and surface drains, and resloping the toe of the slide. At Ridgway Dam, stabilization was achieved mainly by installation of 51 rock anchors.

At Fruitgrowers and Monument Dams, reactivation along landslide failure surfaces in abutments has occurred. As noted above, left-abutment movement at Fruitgrowers Dam impinged on the dam's spillway, which in 1987 was moved to the right end of the embankment. However, movement at Monument Dam thus far has been small enough that no dam distress has occurred. Movement currently is being monitored at both dams.

At other dams, instability has occurred as minor surficial slides, mainly as a result of seepage; these have required maintenance, sometimes on a continuing basis. In a few cases (notably at Fruitgrowers, Parsons, Vega, and Vela Dams), slides interfered with dam spillways, requiring spillway remediation or relocation (Parsons Dam). At Taylor Park Dam, rock fall from cliffs above both abutments continues to be a maintenance problem.

Seepage

Of the 56 Colorado dams constructed on landslides, 29 have encountered unanticipated seepage problems through an abutment or the foundation. Although much of this seepage has been minor, not requiring correction, some has demanded costly engineering remediation, such as grouting, the installation of impervious membranes, cutoff walls, or interceptor trenches, and/or installation of drainage systems to protect the body of the dam. In dams built in the second half of the 20th century, the existence of pre-existing landslides in abutments or foundations was usually recognized before or during construction, and seepage-control measures were installed during construction. These measures have been generally successful in preventing seepage; examples of such dams are Platoro, Ridgway, Vega, and Yamcolo.

MITIGATIVE PROCEDURES

Procedures used to prevent or alleviate problems encountered because of the existence of landslides at potential or actual damsites include:

- Measures that avoid or alleviate the problem as part of the planning process (i.e., *passive* measures). These commonly include avoidance, dam-type selection, control of reservoir level, and/or relocation of the spillway.
- Physical prevention or remediation measures (i.e., *active* measures), such as removal of all or part of the landslide; flattening the slope; construction of berms that serve as buttresses, of impervious membranes or cutoffs, and of retaining walls; installation of surface and/or underground drains; and installation of rock-anchor systems.

Planning Strategies

Avoidance - The basic preventive measure if a landslide has been recognized in a planned foundation or abutment area during the siting process is *avoidance*, i.e., either complete abandonment of the site or removal to a more favorable location nearby. In the early part of the 20th century, this concept was not utilized effectively; however, with today's more rigorous application of the principles of geology, the costs of avoidance are effectively balanced against the costs of mitigative measures.

Substitution of Earth or Rockfill Dams for Concrete Structures - As noted earlier, embankment dams are more flexible than concrete structures, and thus can be built on pre-existing landslides with a lower risk of instability than is the case for concrete, and particularly concrete-arch, structures.

Lowering Reservoir Level or Reducing Rate of Filling - Although use of these procedures tends to impair the function of the dam, they constitute an effective means of increasing abutment stability by reduction of abutment pore pressures and of reducing seepage through the abutment by lowering the head or reducing its rate of increase.

Physical Procedures for Abutment Stabilization

Removal of Landslide Deposits or Flattening of Abutment Slopes - For cases in which a decision has been made to proceed with construction of a dam at the site of a pre-existing landslide, removal of all or part of the landslide material often has been accomplished as a successful preventive measure. Cases in which *all* of the landslide has been removed have not been reported in Table 1. Cases in Table 1 in which most of the landslide has been removed, but some remains as part of the abutment or foundation, include Ridgway, Silver Jack, and Taylor Park Dams.

Earthfill or Rockfill Buttresses - Earthfill and rockfill berms often have been used as buttresses to increase the stability of abutment slopes. In many cases, the material for construction of a berm is obtained directly from the landslide deposits excavated from upslope on the abutment. The best Colorado example of the use of an embankment berm to control an abutment landslide by serving as a buttress is that at Silver Jack Dam.

Dam Serving as a Buttress - Often, potential abutment landslides have been successfully buttressed by the mass of the dam itself. In this manner, the abutment slopes may be more stable than before the dam was built. All dams have a buttress effect; in Table 1, the buttressing effect of Gross Dam has been especially noted.

Concrete Cutoffs or Keys - A concrete cutoff is often placed in a trench that is excavated beneath the location of the dam core. The main function of a cutoff usually is to reduce seepage through the foundation or abutment. However, these cutoffs also “key” the dam into the foundation or abutment, and thus increase stability in addition to reducing permeability.

Retaining Walls - Conventional retaining structures occasionally have been used to increase abutment stability during construction. These walls commonly are left in place and become part of the dam.

Anchors - Anchors (usually prestressed steel tendons) are often used to increase the stability of rock abutments, particularly during construction. This was done successfully on the left abutment of Ridgway Dam.

Gunniting - Although gunniting provides almost no direct structural strength, it occasionally is used to increase stability of very steep slopes by inhibiting the entry of water.

“Dental Work” - “Dental work” is the filling of joints and other voids with cement grout or concrete to increase local stability and to possibly reduce permeability. It is often used in rock abutments during construction.

Reducing Foundation/Abutment Seepage

Impervious Cutoffs - Probably the most common seepage-reduction measures are impervious cutoffs that are constructed through the landslide materials to solid rock. Usually these cutoffs are made of concrete and serve as "keys" to stabilize the structure; however some consist of impervious soil or bentonitic slurries, which do little to increase stability. Cutoffs are commonly installed during the construction process.

Drainage Systems - Drainage systems are commonly used to intercept water before it enters the landslide deposit or to remove existing water from the landslide material. Drainage helps to stabilize landslide materials, to decrease seepage losses through the abutment or foundation, and to reduce the possibility of piping. Drainage systems commonly consist of one or more of the following: surface drainage, interceptor trench drains (vertical trenches backfilled with pervious materials, such as sand and gravel), "horizontal" drains, adits and galleries, filter blankets, and pumped drains. Toe drains and relief wells also are used to allow water to exit the foundation or abutment without building up pore pressures within the embankment mass. Any of these measures can be installed during dam construction as preventive measures, or may be added later as remedial measures.

Impervious Curtains, Membranes, and Blankets - Seepage can also be intercepted and diverted by impervious curtains that have little inherent structural strength, i.e., cement or chemical grout curtains, plastic or geosynthetic membranes, and clay blankets. These measures are not intended to act as strengthening "keys" through the landslide material to bedrock, but may increase stability as well as reduce seepage by locally lowering pore pressures in the foundation and abutments.

SUMMARY AND CONCLUSIONS

Geologists sometimes feel that it is impossible to safely construct a dam on a pre-existing landslide. Conversely, some engineers have been known to assume that today's advanced construction and prevention techniques can overcome *any* landslide problem. This study of 56 Colorado dams that have been built on pre-existing landslides shows that reality is somewhere between these extremes: some dams have been built on landslides with no ensuing difficulties, even if preventive measures have not been used; others have encountered serious, and costly, seepage problems, and a few have been subjected to slope-failure problems. Most of these problems have been at least partly alleviated by installation of remedial measures, such as impervious membranes or drainage systems, but at a cost to the dam owner. *Avoidance* of sites where landslides result in exorbitant costs during construction or remediation should always be considered as a serious option during the process of damsite location.

This survey of Colorado dams indicates that seepage has been the most common negative result of building a dam on a landslide. Seepage occurs through open joints and failure surfaces in rock and earth landslides, and through voids in more-pervious landslide masses, such as rock-fall talus and debris flows. When extreme, seepage can possibly lead to piping (particularly in loose granular materials) and possible dam failure; however, I found no case in which this was even a slight possibility for a dam built on a landslide in Colorado. More commonly, seepage has resulted in loss of water intended for irrigation or electric-power production, thus resulting in dam inefficiency and economic loss.

There seems to be no clear indication as to which landslide types perform best or worst when used as foundations or abutments. In contrast, most types have proven to be fairly stable, with slides in shales probably performing the poorest. In contrast, all landslide types seem to be subject to seepage problems unless preventive or remedial measures are taken. Generally speaking, rock-fall (e.g., talus) and debris-flow deposits are fairly pervious, and thus provide ready paths for seepage; however, some of these deposits include enough fine material to be relatively impervious. Thus, in regard to both stability and seepage, the physical characteristics of the individual landslides and the materials of which they are comprised should be carefully considered in the siting process for any dam in which a landslide will be part of the foundation or an abutment. Of particular importance is the permeability of the landslide mass.

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Table 1. Colorado dams on landslides
Unit conversions: 1 ft = 0.305 m; 1 ac ft = 1233.5 m^3

Dam name/ River or stream	County/ Latitude, longitude	Dam type/ Purpose	Year constr./ Owner	Height/ Length, ft	Storage volume, ac ft	Landslide type	Landslide position at dam	Comments	References/Sources of information
Atkinson/ Atkinson Creek	Mesa/ 39.1000, 107.8833	Earthfill/ Irrigation, hydro-electric	1893/ Federal gov't.	35/ 750	2000	Massive rock slide	Entire dam and reservoir	Entire dam and reservoir are on Grand Mesa landslide complex (mainly basalt on claystone, mantled by glacial deposits). Seepage from both abutments has been a problem (1987).	Yeend (1969, 1973), Colton et al. (1975c); Tweto et al. (1978); Baum & Odum (1996, 2003)/ U.S. Bureau Reclamation; CO Div. Water Resources
Beaver/ Minnesota Creek tributary	Gun-nison/ 38.8217, 107.4500	Earthfill/ Irrigation, fire protection, recreation	1958/ Private	113/ 870	1850	Shear, collapse of sedi-mentary rocks	Both abutments	Abutments/foundation of sandstone, shale, and coal. Coal burned naturally, reducing support for overlying layers, which failed locally. Abutment leakage led to 1997 installation of 60-mil impervious liner at both ends of dam.	Ellis et al. (1987)/ Consultants' reports; CO Div. Water Resources
Beaver Park/ Beaver Creek	Rio Grande/ 37.5983, 106.6667	Earthfill/ Recreation, fisheries, irrigation	1912/ State gov't.	114/ 435	5800	Apparent rock slide	Right abutment	Abutment, composed of badly fractured latite tuff mapped as "landslide" by Atwood (1918), has been stable, but has leaked considerably through the years.	Atwood (1918)/ CO Div. Water Resources; U.S. Army Corps Engineers
Big Beaver/ Bull Creek tributary	Mesa/ 39.0833, 108.0333	Earthfill/ Irrigation	1936/ Private	37/ 180	175	Massive rock slide	Entire dam and reservoir	Entire dam and reservoir on Grand Mesa landslide complex (mainly basalt on claystone, mantled by glacial deposits). Seepage through foundation in 1992. Seepage both north and south of dam.	Yeend (1969, 1973); Colton et al (1975e); Baum & Odum (1996, 2003)/ CO Div. Water Resources
Black Lake #1/ Black Gore Creek	Eagle/ 39.5428, 106.2200	Earthfill/ Water supply, recreation	1939 (rebuilt 1995)/ Local gov't.	72/ 400	327	Debris flow from ancient landslide	Right abutment	Dam was rebuilt in 1995. Much of original landslide was removed. Interceptor trench to sound bedrock. Grout curtain to competent bedrock in foundation and right abutment.	---- / CO Div. Water Resources; consultant's report
Bonham/ Big Creek	Mesa/ 39.1028, 107.9000	Earthfill/ Irrigation, hydro-	1900/ Federal gov't.	38/ 1500	1959	Massive rock slide	Entire dam and reservoir	Entire dam and reservoir on Grand Mesa landslide complex (mainly basalt on claystone, mantled by glacial deposits). Minor	Yeend (1969, 1973); Colton et al. (1975c); Tweto et al. (1978);

Dam name/ River or stream	County/ Latitude, longitude	Dam type/ Purpose	Year constr./ Owner	Height/ Length, ft	Storage volume, ac ft	Landslide type	Landslide position at dam	Comments	References/Sources of information
		electric						foundation seepage.	Baum & Odum (1996, 2003)/ U.S. Bureau Recl.amation
Burnt Mesa/ S. Branch Hunt Creek tributary	Routt/ 40.1333, 107.0183	Earthfill/ Irrigation	1957/ Private	40/ 273	212	Slide in Mancos Shale	Right abutment	Right abutment is in Mancos Shale slide. In 1987, seepage in right abutment area. However, landslide generally has had no negative effect on dam.	Colton (1975f); Madole (1989)/ CO Div. Water Resources
Cedar Mesa/ Surface Creek	Delta/ 39.0483, 107.8483	Earthfill/ Irrigation	1944/ Private	47/ 1250	1160	Massive rock slide	Entire dam and reservoir	Entire dam/ reservoir on Grand Mesa landslide complex (mainly basalt on claystone, mantled by glacial deposits). "Blowout" (1971) in foundation at left abutment corrected by vinyl membrane. Satisfactory performance since.	Yeend (1969, 1973); Colton et al. (1975c)/ CO Div. Water Resources; Consultant's report
Clear Lake/ South Clear Creek	Clear Creek/ 39.6717, 105.7017	Rockfill/ Hydro-electric, recreation	1914/ Public utility	40/ 180	400	Rock slide	Entire dam foundation and both abutments	Dam was built on landslide dam formed by rock slides (much talus) from both valley walls. Seepage and piping occurred through left abutment talus, resulting in "sinkholes (1997). Remedial grouting employed in talus.	Widmann & Miersemann (2001); Hammer (2002)/ CO Div. Water Resources
Currier #2/ Buzzard Creek	Mesa/ 39.2917, 107.7233	Earthfill/ Irrigation, fire protection, stock	1968/ private	44/ 368	320	Land-slide in shale/ sand-stone	Left abutment	Left abutment is in "young landslide," which is in "state of metastable equilibrium" (Soule, 1988). In 1993-94, slide activity on left abutment encroached on emergency spillway; problem solved by diverting water from distressed area. No other problems.	Soule (1988)/ CO Div. Water Resources; U.S. Soil Cons. Service
D.D.&E. Wise/ Hulch Ck. Diversion Ditch	Rio Blanco/ 40.1667, 107.6667	Earthfill/ Irrigation	1946/ Private	41/ 340	1244	Massive slide in shale	Entire dam and reservoir	Aldrich Lake and D.D. & E. Wise Dam are located on eastern edge of massive 5-mi^2 landslide in Mancos Shale. Landslide material has had no negative effect on dam or reservoir performance.	Colton (1975f); Tweto (1976); Reheis (1984); Madole (1989)/ CO Div. Water Resources
Eggleston/ Kiser Creek	Delta/ 39.0400, 107.9483	Earthfill/ Irrigation	1949/ Private	40/ 330	3460	Massive rock slide	Entire dam and reservoir	Entire dam and reservoir are on Grand Mesa landslide complex (mainly basalt on claystone, mantled by glacial deposits). Very minor seepage; no serious problems.	Yeend (1969, 1973); Colton et al. (1975c); Tweto et al. (1978); Baum & Odum (1996, 2003)/ CO Div. Water Resources

Dam name/ River or stream	County/ Latitude, longitude	Dam type/ Purpose	Year constr./ Owner	Height/ Length, ft	Storage volume, ac ft	Landslide type	Landslide position at dam	Comments	References/Sources of information
Fruit-growers/ Alfalfa Run Creek	Delta/ 38.8278, 107.9550	Earthfill/ Irrigation, recreation	1939/ Federal gov't.	55/ 1520	7548	Large block-glide landslide in shale	Left abutment and part of left foundation	Slow reactivation along gently dipping failure surface in Mancos Shale has occurred since construction of the dam, presumably tilting original spillway, which was replaced in 1987. Movement currently is being monitored.	----/ U.S. Bureau Reclamation
Good-enough #2/ Leroux Creek tributary	Delta/ 39.0383, 107.6800	Earthfill/ Irrigation, water supply	1928/ Private	40/ 760	1077	Massive rock slide	Entire dam and reservoir	Entire dam and reservoir are on Grand Mesa landslide complex (mainly basalt on claystone, mantled by glacial deposits). Minimal foundation leakage at toe.	Yeend (1969, 1973); Colton et al. (1975c); Baum & Odum (1996, 2003)/ CO Div. Water Resources
Granby #12/ Dirty George Creek tributary	Delta/ 38.9983, 108.0417	Earthfill/ Irrigation	1949/ Private	33/ 895	909	Massive rock slide	Entire dam and reservoir	Entire dam and reservoir are on Grand Mesa landslide complex (mainly basalt on claystone, mantled by glacial deposits). Foundation seepage during 1970's caused small slumps in toe of embankment. Minor foundation seepage still occurs.	Yeend (1969, 1973); Colton et al. (1975e); Ellis & Gabaldo (1989); Baum & Odum (1996, 2003)/ CO Div. Water Resources
Gross/ South Boulder Creek	Boulder/ 39.9483, 105.3583	Concrete gravity/ Water supply, hydro-electric, recreation	1955/ Public utility	340/ 1090	40,990	Deep-seated rock slide	Both abutments	Wahlstrom (1974, p. 74) noted deep-seated "gravity-slip surfaces in steep-walled canyon in massive crystalline rock." No effect on dam performance. Dam has buttressed any possible future movement.	Wahlstrom (1974/ ----
Hahns Peak/ Willow Creek	Routt/ 40.8350, 106.9850	Earthfill/ Recreation, fish & wildlife	1978/ State gov't.	38/ 267	698	Slide in shale	Left abutment	Slide in shale of Morrison Formation serves as left abutment. No problems have developed.	Madole (1991b)/ CO Div. Water Resources
Hogchute/ Kannah Creek	Mesa/ 38.9950, 108.1117	Earthfill/ Water supply	1947/ City gov't.	56/ 620	765	Rock slide	Entire dam and reservoir on landslide	Entire dam and reservoir on part of Grand Mesa slide complex. Right abutment particularly is on basalt talus. No negative effects on dam.	Yeend (1969,1973); Colton et al. (1975d); Ellis & Gabaldo (1989)/ CO Div. Water Resources
Joe Wright/ Joe Wright Creek	Larimer/ 40.5600, 105.8700	Earthfill/ Water supply, irrigation, recreation	1979/ City gov't.	148/ 2300	9353	Massive rock slump	Right abutment and right end of dam	Entire right valley wall is huge Quaternary landslide mass (volcanics overlying sedimentary rocks), probably formed upon glacier retreat. No problems.	Colton et al. (1975b)/ ----

Dam name/ River or stream	County/ Latitude, longitude	Dam type/ Purpose	Year constr./ Owner	Height/ Length, ft	Storage volume, ac ft	Landslide type	Landslide position at dam	Comments	References/Sources of information
Jones #2/ Sheep Creek tributary	Grand/ 40.0600, 106.4517	Earthfill/ Water supply, irrigation	1887/ Private	40/ 325	463	Massive block glide	Entire dam and reservoir	Entire dam and reservoir lie on 5-mi^2 glide block in Niobrara and Dakota shales and sandstones. Abutments are in “fractured shales;” seepage from toe drains. Reservoir serves as water supply for City of Kremmling.	Barclay (1968, p. 157); Izett & Barclay (1973); Colton (1975f); Madole (1991a)/ CO Div. Water Resources
Kehmeier/ Surface Creek tributary	Delta/ 39.0583, 107.8350	Earthfill, irrigation	1949/ Private	33/ 564	358	Massive rock slide	Entire dam and reservoir	Entire dam and reservoir on Grand Mesa landslide complex (mainly basalt on claystone, mantled by glacial deposits). Minor seepage from both abutments.	Yeend (1969, 1973); Colton et al. (1975c); Tweto et al. (1978)/ CO Div. Water Resources
Kennicott Slough/ Kiser Creek tributary	Delta/ 39.0267, 107.9567	Earthfill/ Irrigation	1946/ Private	38/ 1246	1034	Massive rock slide	Entire dam and reservoir	Entire dam and reservoir on Grand Mesa landslide complex (mainly basalt on claystone, mantled by glacial deposits). Minor seepage from left abutment; performance generally good.	Yeend (1969, 1973); Colton et al. (1975c); Tweto et al. (1978); Baum & Odum (1996, 2003)/ CO Div. Water Resources
Kiser Slough/ Kiser Creek	Delta/ 39.0236/ 107.9481	Earthfill/ Irrigation, recreation	1950/ Private	38/ 1050	652	Massive rock slide	Entire dam and reservoir	Entire dam and reservoir on Grand Mesa landslide complex (mainly basalt on claystone, mantled by glacial deposits). Major foundation leakage; drainage trenches added as remedial measure have improved situation.	Yeend (1969, 1973); Colton et al. (1975c); Tweto et al. (1978); Baum & Odum (1996, 2003)/ CO Div. Water Resources
Knox/ Surface Creek tributary	Delta/ 39.0417, 107.8783	Earthfill/ Irrigation	1954/ Private	40/ 512	335	Massive rock slide	Entire dam and reservoir	Entire dam and reservoir on Grand Mesa landslide complex (mainly basalt on claystone, mantled by glacial deposits). Minor seepage from both abutments.	Yeend (1969, 1973); Colton et al. (1975c); Tweto et al. (1978); Baum & Odum (1996, 2003)/CO Div. Water Resources
Lower Cogdill/ Govern-ment Corral Creek	Moffatt/ 40.9717, 107.3283	Earthfill/ Water supply, fish & wildlife, fire protection	1956/ Private	39/ 480	275	Massive slide in shale/ sand-stone	Entire dam and reservoir	Entire dam and reservoir lie in massive (7-mi^2) landslide area overlying Lewis Shale and Mesaverde Group (sandstone & shale). Landslide material has had no negative effect on dam or reservoir performance.	Colton (1975f); Madole (1982)/ CO Div. Water Resources

Dam name/ River or stream	County/ Latitude, longitude	Dam type/ Purpose	Year constr./ Owner	Height/ Length, ft	Storage volume, ac ft	Landslide type	Landslide position at dam	Comments	References/Sources of information
Matheson/ Trouble-some Creek	Grand/ 40.2950, 106.2917	Earthfill/ Irrigation	1951/ Private	60/ 192	1570	Massives slide in Tertiary volcanics	Left abutment and possibly foundation	Dam was built on NE edge of large landslide area in Tertiary volcanics overlying Morrison Fm. Seepage in right abutment area through "shattered lava." Clay blanket installed to prevent seepage, with little success.	Madole (1991a)/ CO Div. Water Resources
McElroy/ Pass Creek	Grand/ 40.1383, 106.4717	Earthfill/ Irrigation	1931/ Private	50/ 250	355	Earth flow	Right abutment	Earth flow from Dakota Fm. shales originally dammed Pass Creek. Serves as right abutment and part of foundation. Generally very little seepage; minor seepage at right groin in 1975.	Izett & Barclay (1973); Colton (1975f); Madole (1991a)/ CO Div. Water Resources
McKelvie #1/ Plateau Creek tributary	Mesa/ 39.2117, 107.7517	Earthfill/ Irrigation	1943/ Private	36/ 450	357	"Old land-slide"	Entire dam	Entire dam is in area mapped by Soule (1988) as "old landslide." Minimal foundation seepage (slightly boggy downstream; no flowing water). Dam performance good.	Soule (1988)/ CO Div. Water Resources
McKoon/ Young's Creek)	Delta/ 39.0400, 107.9250	Earthfill/ Irrigation	1948/ Private	33/ 275	203	Massive rock slide	Entire dam and reservoir	Entire dam and reservoir are on Grand Mesa landslide complex (mainly basalt on claystone, mantled by glacial deposits). No negative effects on dam.	Yeend (1969, 1973), Colton et al. (1975c), Tweto et al. (1978); Baum & Odum (1996, 2003)/CO Div. Water Resources
McMahon #2/ Red Dirt Creek	Grand/ 40.1783, 106.5717	Earthfill/ Irrigation	1945/ Private	50/ 987	4570	Slump–earth flow	Left abutment	Left abutment is slump–earth flow in Morrison Fm. shale and sandstone. Landslide apparently has had no negative effect on dam performance.	Madole (1991a)/ CO Div. Water Resources
Milk Creek/ Milk Creek	Grand/ 40.2717, 106.5617	Earthfill/ Irrigation	1925/ Private	36/ 140	164	Massive slide in shale/ sand-stone	Entire dam and reservoir	Entire dam and reservoir lie in large landslide area overlying Benton Shale and Dakota Sandstone. Landslide material has had no negative effects on dam performance.	Colton et al. (1975f); Madole (1991a)/ CO Div. Water Resources
Monument/ Minnesota Creek tributary	Gun-nison/ 38.8817, 107.4717	Earthfill/ Irrigation	1889/ Private	76/ 422	632	React-ivated debris slide	Left abutment	Slide extends vertically about 400 ft above left end of dam and 1500 ft laterally; consists mainly of sandstone cobbles and boulders in clay matrix. Instrumented since 1992 to record movement; has been no distress. No significant remedial measures have been installed.	Colton et al (1975g); Ellis et al. (1987); Norfleet & Marvin (1995)/ CO Div. Water Resources

Dam name/ River or stream	County/ Latitude, longitude	Dam type/ Purpose	Year constr./ Owner	Height/ Length, ft	Storage volume, ac ft	Landslide type	Landslide position at dam	Comments	References/Sources of information
Monument #1/ Monument Creek	Mesa/ 39.1083, 107.7500	Earthfill/ Irrigation	1960/ Private	35/ 500	760	Massive rock slide	Entire dam and reservoir	Entire dam and reservoir on Grand Mesa landslide complex (mainly basalt on claystone, mantled by glacial deposits). Some seepage from left abutment.	Yeend (1969, 1973), Colton et al (1975c)/ CO Div Water Resources
Mountain Home/ Trinchera Creek	Costilla/ 37.3933, 105.3933	Earthfill/ Irrigation, recreation, fish & wildlife	1906/ Private	153/ 475	25,992	Rock-block slide	Left abutment	Translational slide in basalt underlain by Tertiary Santa Fe Fm. siltstone and sandstone. No movement since construction. Because overflow spillway crosses slide block, possible instability was conjectured; however, 1993 analysis indicated no instability problems.	----/ CO Div. Water Resources, consultants' reports
North Michigan Creek/ Michigan River	Jackson/ 40.5483, 106.0217	Earthfill/ Recreation, fish & wildlife	1963/ State gov't.	74/ 465	2546	Earth flow	Left (and, possibly, right) abutment	Left abutment (and possibly right abutment) is on landslide masked by glacial deposits. Except for minor right-abutment seepage, landslide has had no negative effect on dam.	Madole (1991b)/ CO Div. Water Resources
Overland #1/ Cow Creek (Muddy Creek tributary)	Delta/ 39.0783, 107.6450	Earthfill/ Irrigation	Origin-ally early 1900's; rebuilt 1987/ Private	85/ 3200	8208	Massive rock slide	Entire dam and reservoir	Entire dam and reservoir are on Grand Mesa landslide complex (mainly basalt on claystone, mantled by glacial deposits). Before1987, there was toe-buttress instability, possibly due to landslide material. Toe-buttress drains were installed and material added to toe buttress.	Yeend (1969, 1973), Colton et al. (1975c)/ CO Div. Water Resources; consultant's report
Park/ Surface Creek	Delta/ 39.0467, 107.8750	Earthfill/ Irrigation	1940/ Private	46/ 750	3940	Massive rock slide	Entire dam and reservoir	Entire dam and reservoir are on Grand Mesa landslide complex (mainly basalt on claystone, mantled by glacial deposits). Slide material is fragmental basalt and fairly permeable. Thus, there have been seepage problems (20-30 gpm through base of left abutment). Bentonite added to abutment in 1997 to slow seepage.	Yeend (1969, 1973), Colton et al. (1975c), Tweto et al. (1978); Baum & Odum (1996, 2003/CO Div. Water Resources
Parsons/ Carter Creek	Grand/ 40.2683, 106.4050	Earthfill/ Irrigation, stock	1952/ Private	39/ 275	140	Slide in Pierre Shale	Left abutment & spillway	Left abutment and original spillway in toe of pre-existing Pierre Shale slide. Spillway offset by slide. Some seepage still flows from slide.	Colton et al. (1975f), Madole (1991a)/ CO Div. Water Resources, consultant's reports

Dam name/ River or stream	County/ Latitude, longitude	Dam type/ Purpose	Year constr./ Owner	Height/ Length, ft	Storage volume, ac ft	Landslide type	Landslide position at dam	Comments	References/Sources of information
Platoro/ Conejos River	Conejos/ 37.3492, 106.5433	Rockfill/ Irrigation	1951/ Federal gov't.	165/ 885	67,790	Massive slump block	Left abutment	Left abutment is toe of huge (length>0.5 mi) andesite (quartz latite?) slump block. Abutment is stable. No seepage problems.	----/ U.S. Bureau Reclamation
Poose Creek/ Poose Creek	Rio Blanco/ 40.1317, 107.2583	Earthfill/ Recreation, fish & wildlife	1969/ State gov't.	41/ 435	544	Slide in volcanic rock, shale, glacial deposits	Right abutment (possibly entire dam)	Dam was built on mixture of landslide/glacial deposits left behind as glacier retreated from valley. Landslide is mixture of shale/sandstone from Brown's Park, Morrison, and Dakota Fms., of overlying Tertiary volcanics, and glacial deposits. No negative effects on dam.	Colton et al. (1975f), Madole (1989)/ CO Div. Water Resources
Ridgway/ Uncompah-gre River	Ouray/ 38.1500, 107.7500	Earthfill/ Irrigation, water supply, recreation, flood control	1987/ Federal gov't.	330/ 2430	89,230	Slides in siltstone, sand-stone, mud-stone	Left abutment	Reactivation of slides in left abutment Dakota Group and Morrison Fm. sandstones, silt-stones, and mudstones during construction; most slide debris removed. Incipient move-ment of large slide block remedied by instal-lation of: (1) 51 rock anchor tendons, (2), tunnel drains, (3) monitoring instrumentation. Abutment currently is stable; no seepage.	Von Thun (1987)/ U.S. Bureau Reclamation reports
Rio Grande/ Rio Grande River	Hinsdale / 37.7206, 107.2667	Earthfill/ Irrigation, recreation	1916/ Private	116/ 600	52,192	Rock slide in andesite	Left abutment	Huge andesite slide at, and above, left abut-ment (Atwood, 1918). Abutment seepage led to remedial measures in late 1990's: (1) retaining wall, (2) horizontal and French drains. Installation of piezometers and inclinometers.	Atwood (1918); Atwood & Mather (1932, p. 157)/ CO Div. Water Resources; consultants' reports
Scholl/ Corral Creek	Grand/ 40.1367, 106.2000	Earthfill/ Irrigation	1964/ Private	59/ 180	549	Rock slide	Right abutment	Right end of dam abuts against slide of basalt boulders up to 6 ft in diameter derived from basalt flows in upper valley wall. Serious seepage problems through this pervious mass. In 1964, right groin was grouted. In 1965, 1972, 1989-90, impermeable membranes were placed in right abutment, but seepage continued; "sinkholes" formed on surface.	Colton et al. (1975d); Madole (1991a)/ CO Div. Water Resources

Dam name/ River or stream	County/ Latitude, longitude	Dam type/ Purpose	Year constr./ Owner	Height/ Length, ft	Storage volume, ac ft	Landslide type	Landslide position at dam	Comments	References/Sources of information
Sheriff/ Trout Creek	Rio Blanco/ 40.1483, 107.1367	Earthfill/ Irrigation, water supply	1955/ Local gov't.	67/ 630	1450	Slide in volcanic rock, shale, glacial deposits	Right abutment (possibly entire dam)	Dam built on mixture of landslide and glacial deposits left behind as glacier retreated from valley. Landslide is mixture of shale/ sandstone from Brown's Park and Morrison Fms., of overlying Tertiary volcanics, and glacial deposits. No negative effects on dam.	Colton et al. (1975f); Madole (1989)/ CO Div. Water Resources
Silver Jack/ East Fork Cimarron River	Gun-nison/ 38.2450, 107.5433	Earthfill/ Irrigation, recreation	1971/ Federal gov't.	173/ 1050	15,363	Large slides in Mancos Shale and glacial debris	Both abutments	Excavation for right end of dam was made in toe of slide causing 500,000-yd^3 reactivation. Remedial works: regrading slide, redesigning dam to avoid further cutting of slide toe, re-locating spillway, horizontal/surface drains, 115,300-yd^3 buttress fill. Measures were successful; abutment currently is stable.	Logan & Davis (1973); Colton et al. (1975g)/ U.S. Bureau Reclamation reports
Stillwater #1/ Bear River	Garfield/ 40.0300, 107.1200	Earthfill/ Irrigation	1939/ Private	89/ 1500	7410	Slide in volcanic rock, shale, glacial deposits	Left abutment	Left abutment is partly on volcanic bedrock, partly on toe of landslide, which is mixture of shale/sandstone form Brown's Park and Morrison Fms., Tertiary volcanics, and glacial deposits. No problems.	Colton et al. (1975f); Madole (1989)/ CO Div. Water Resources
Taylor Park/ Taylor River	Gunni-son/ 38.8050, 106.5950	Earthfill/ Irrigation, recreation	1937/ Federal gov't.	206/ 616	111, 260	Rock fall	Right abutment	Right abutment covered by thick talus cone from sedimentary rock cliffs. Talus removed during construction except for area 75 ft downstream from centerline of dam to down-stream toe, thus posing a very slight piping threat at downstream toe. Continuing rock-fall activity from cliffs above both abutments.	----/ U.S. Bureau Reclamation reports
Trujillo Meadows/ Los Piños River	Conejos/ 37.0050, 106.4500	Earthfill/ Recreation, fish & wildlife	1956/ State gov't.	50/ 163	1263	Rock slide in tuffs and ash flows	Left abutment and foundation	Dam built on toe of large Quaternary rock slide that had blocked the river. No stability problems, but in 1990's seepage through left abutment became unacceptable. Impervious cutoff placed in left abutment in 1998.	Colton et al. (1975a); Lipman (1975); Hollingsworth & Hollingsworth (1997)/ CO Div. Water Resources; U.S. Forest Service; consultant's report

Dam name/ River or stream	County/ Latitude, longitude	Dam type/ Purpose	Year constr./ Owner	Height/ Length, ft	Storage volume, ac ft	Landslide type	Landslide position at dam	Comments	References/Sources of information
Upper Stillwater/ Bear River	Garfield/ 40.0433, 107.0717	Earthfill/ Recreation, fish & wildlife	1965/ State gov't.	40/ 275	902	Slide in volcanic rock, shale, glacial deposits	Entire dam in landslide/ glacial deposits	Valley walls are covered by massive Quaternary landslides, mixtures of sandstone/shale from Brown's Park and Morrison Fms., Tertiary volcanics, and glacial deposits. At damsite, landslide and glacial deposits are intermingled. No dam problems related to geology.	Colton et al. (1975f); Madole (1989/ CO Div. Water Resources
Vega/ Plateau Creek	Mesa/ 39.2258, 107.8150	Rockfill-earthfill/ Irrigation, recreation	1959/ Federal gov't.	162/ 2100	38,102	"Old land-slide" & "ancient debris flow"	Both ends of dam	Soule (1986, 1988) mapped left abutment as "old landslide" and right abutment as "ancient debris flow complex." Both are generally stable. However, local reactivation has occurred on both abutments since construction; damage was done to spillway. No current activity.	Colton et al. (1975c); Soule (1986, 1988)/ U.S. Bureau Reclamation
Vela/ Surface Creek tributary	Delta/ 39.0633 107.8733	Earthfill/ Irrigation	1959/ Private	39/ 625	517	Massive rock slide	Entire dam and reservoir	Entire dam and reservoir are on Grand Mesa landslide complex (mainly basalt on claystone, mantled by glacial deposits). Minor foundation seepage. Local spillway slide problems.	Yeend (1969, 1973); Colton et al. (1975c); Baum & Odum (1996, 2003)/ CO Div. Water Resources
Ward Creek/ Ward Creek	Delta/ 39.0133, 107.9967	Earthfill, irrigation, recreation	1957, private	47/ 880	482	Massive rock slide	Entire dam and reservoir	Entire dam and reservoir are on Grand Mesa landslide complex (mainly basalt on claystone, mantled by glacial deposits).	Yeend (1969, 1973); Colton et al. (1975c); Baum & Odum (1996, 2003)/ CO Div. Water Resources
Whiteley Peak/ Diamond Creek	Grand/ 40.3283, 106.5167	Earthfill/ Irrigation, water supply	1952/ Private	62/ 775	1095	Shale slide	Entire dam	Slides in Pierre Shale form both abutments and foundation of dam. Reactivation of part of right-abutment slide in 1986 slightly damaged spillway. No other serious effects.	Hail (1968); Colton (1975f; Madole (1991a)/ CO Div. Water Resources
Yamcolo/ Bear River	Garfield/ 40.0550, 107.0467	Earthfill/ Irrigation, water supply	1980/ Local gov't.	110/ 1900	12,124	Rock slide and debris flow	Both abutments and part of foundation	Right end of dam is on massive Quaternary rock slide (mixture of sandstone/shale from Brown's Park and Morrison Fms., Tertiary volcanics, glacial deposits); left end and part of foundation are on Quaternary debris flow. No problems.	Colton et al. (1975f); Madole (1989)/CO. Geol. Survey; CO Div. Water Resources

Dam name/ River or stream	County/ Latitude, longitude	Dam type/ Purpose	Year constr./ Owner	Height/ Length, ft	Storage volume, ac ft	Landslide type	Landslide position at dam	Comments	References/Sources of information
Young's Creek Nos. 1 and 2/ Young's Creek tributary	Delta/ 39.0383, 107.9117	Earthfill/ Irrigation	1952/ Private	57/ 505	795	Massive rock slide	Entire dam and reservoir	Entire dam and reservoir are on Grand Mesa landslide complex (mainly basalt on claystone mantled by glacial deposits). Some abutment seepage. Left abutment has been grouted (with little success). Local membrane blanketing without long-term success. Dam is stable.	Yeend (1969, 1973); Colton et al. (1975c); Tweto et al. (1978); Baum & Odum (1996, 2003)/CO Div. Water Resources
YT Ranch/ Grove Creek	Mesa/ 39.1867, 107.8933	Earthfill/ Recreation, irrigation	1911/ Private	38/ 900	185	"Old debris flow"	Entire dam and reservoir	Entire dam and reservoir are on "old debris flow" that originated on summit of Grand Mesa (Soule, 1988). Dam is stable, but has considerable seepage through foundation.	Colton (1975c); Soule (1988)/ CO Div. Water Resources

RETROGRESSIVE SLUMPING AT GRAND MESA, COLORADO

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Key Terms: landslide, slump, retrogressive failure, slope stability

ABSTRACT

Hundreds of Pleistocene-age slump blocks surround Grand Mesa, a 10,500-ft (3200-m)-high, flat-topped mountain in western Colorado. The huge slump blocks (or Toreva blocks) that surround Grand Mesa are among the best-preserved examples of retrogressive slump deposits in the western United States. The blocks exist in all stages of evolution, from incipient forms at the edge of the mesa, to old, strongly tilted, degraded forms at the edges of a landslide bench surrounding the mesa. A sequence of Miocene basalt flows, 200-800 ft (60-240 m) thick, caps the mesa, and Miocene or Oligocene claystone and gravel deposits underlie the basalt. The weak claystone (angle of residual friction between 5° and 8°) tends to spread under the weight of the overlying basalt, thus inducing tension in the basalt. Consequently, large slump blocks have initially detached from the mesa by lateral spreading. After the blocks separated, shear failure of the claystone resulted in backward rotation of the blocks. The back-rotating blocks pushed adjacent, older blocks down a gently inclined ramp. The style of movement evident from the distribution and orientation of blocks is consistent with a listric failure surface that slopes steeply at the head of each slide (beneath the highest block), turns through a relatively tight curve near the base of the block and flattens to a gently sloping ramp beneath older blocks on a landslide bench that surrounds the mesa.

INTRODUCTION

Landslide deposits that have resulted from retrogressive, rotational sliding are widespread, but the mechanisms of these slides have received relatively little attention. The backward rotation of successive blocks differs from the lateral spreading typically associated with retrogressive failure (Voight, 1973; Hansen, 1965). Huge slump blocks (or Toreva blocks) surrounding Grand Mesa, Mesa and Delta Counties, western Colorado, are among the best preserved examples of retrogressive slump deposits in the western United States (Figure 1). Grand Mesa is surrounded by slump blocks that have formed by failure of the mesa's basalt cap rock and underlying claystone. The blocks exist in all stages of evolution, from incipient forms at the edge of the mesa, to old, strongly tilted, degraded forms at the edges of a landslide bench surrounding the mesa (Figure 2). In what follows, we summarize the distribution, morphology, and structure of slump blocks at Grand Mesa as determined by interpretation of aerial photographs and fieldwork conducted during the summers of 1993 and 1994 (Baum and Odum, 1996). We also provide additional observations about the failure mechanisms and shear strength of the claystone, and summarize our analyses of slump-block kinematics.

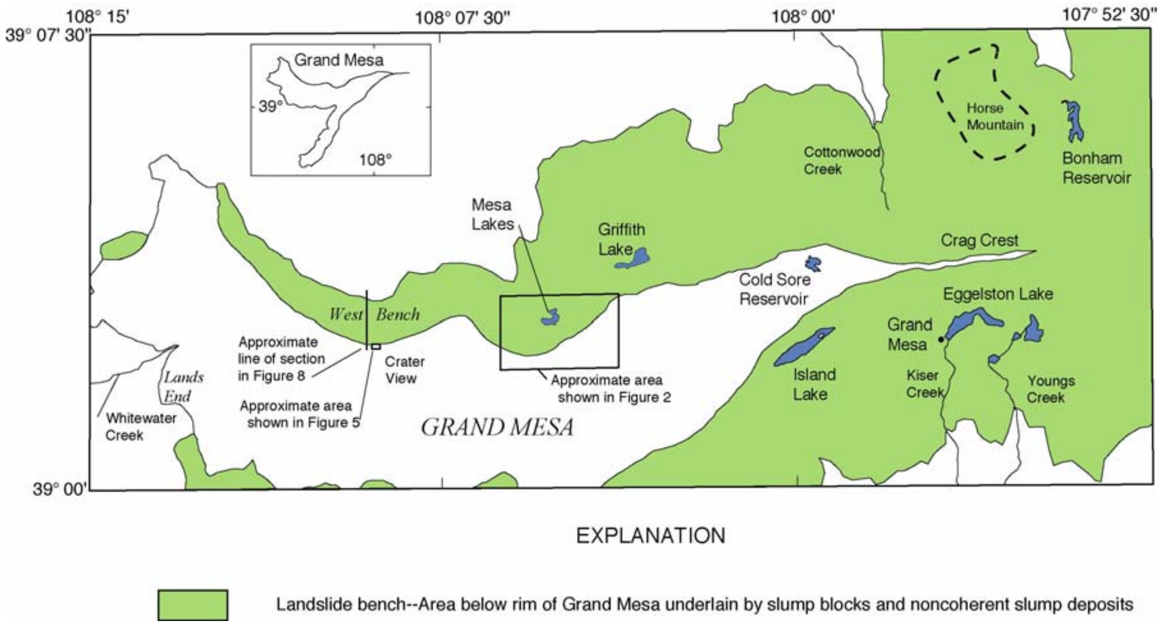


Figure 1. Map showing location of study area and names of geographic features mentioned in the text (modified from Baum and Odum, 1996).

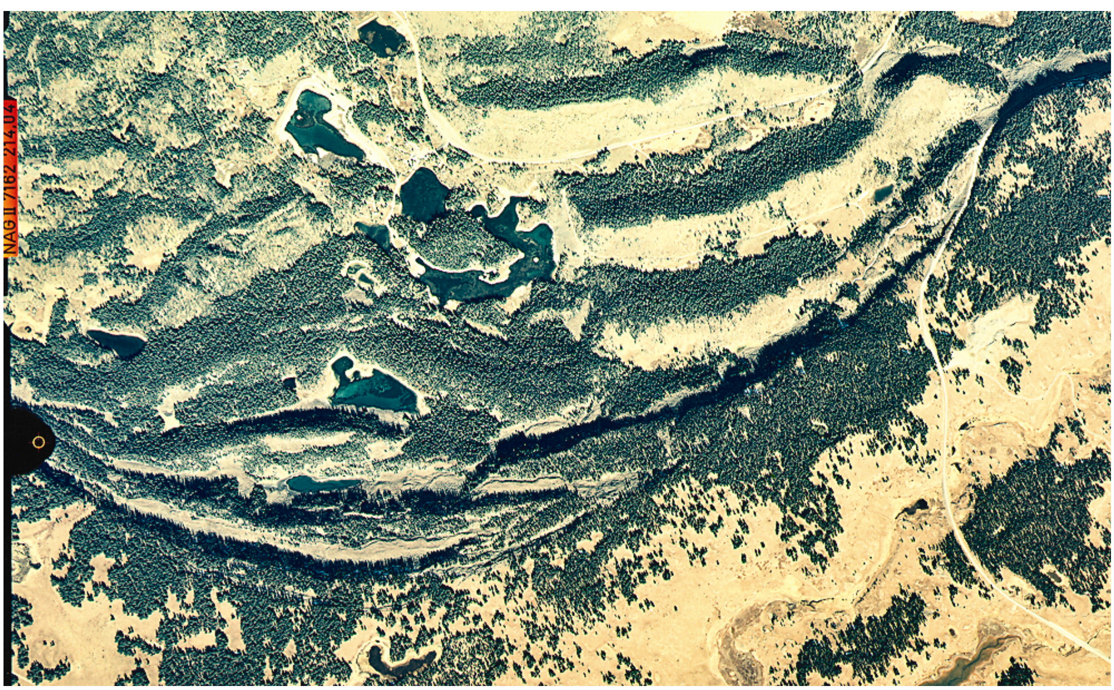


Figure 2. Aerial photograph showing distribution of slump blocks in the Mesa Lakes area, on the north side of Grand Mesa (USDA Forest Service Photograph 1388-6, 29 September 1988).

Previous Work

Several descriptive works have been devoted to retrogressive slump blocks and a few descriptive and analytical works have documented the mechanisms of retrogressive translatory slides. Several workers have mapped and described Toreva blocks, or rotational landslide blocks, from sites in the western U.S., including the Hopi Reservation in Arizona (Reiche, 1937), the Vermillion and Echo Cliffs in northern Arizona (Strahler, 1940), western New Mexico (Moore, 1990a, 1990b), the Diablo Plateau in Texas (Trace, 1942), Grand Mesa, Colorado, (Yeend, 1969) and southern Utah (Machette et al., 1984). Maps and photographs published by these workers indicate that most rotational landslide blocks occur as members of tandem groups, consistent with a retrogressive failure mechanism.

Most previous studies of Grand Mesa have concentrated on bedrock and glacial geology. A. C. Peale described the topography, drainage, and general geology of the area (Hayden, 1876). Henderson (1923), Nygren (1935), and Retzer (1954) reported on various aspects of the glaciation of Grand Mesa, and Nygren (1935) noted the glacial modification of some slump blocks below the rim of Grand Mesa. During the early 1960s, J.R. Donnell of the U.S. Geological Survey mapped the bedrock geology as part of a project to assess oil-shale resources of the Piceance basin (J.R. Donnell & W.E. Yeend, unpublished mapping, 1961-64; Donnell, 1969). At the same time, W.E. Yeend mapped the surficial (Quaternary) geology of Grand and Battlement Mesas (Yeend, 1969). Since then, several workers have incorporated the mapping of Donnell and Yeend (unpublished mapping, 1961-64) into small-scale maps of the area (Tweto et al., 1978; Ellis & Freeman 1984; Ellis et al., 1987; and Ellis & Gabaldo, 1989). Yeend (1969, 1973) described slump blocks from Grand Mesa, showed their distribution on a small-scale map, determined that most predate the last glaciation, reported on the general causes of the slump blocks, and monitored the movement of several incipient slump blocks. Cole & Sexton (1981) summarized the Quaternary stratigraphy of Grand Mesa. Baum & Odum (1996) mapped the slump blocks (1:24,000) and described their distribution and geomorphology. Schuster (this volume) describes a number of dams built on the landslide deposits of Grand Mesa.

SETTING

Geology

Grand Mesa is a high, flat-topped mountain in western Colorado. The mesa is capped by a thick sequence of basalt flows. Slumping at the edge of the mesa has created a broad, gently sloping landslide bench that surrounds the basalt cap (Figures 1 and 3). The study area is in the southern part of the Piceance basin and within the northeast part of the Colorado Plateau physiographic province. Upper Cretaceous and lower Tertiary (Paleocene through upper Eocene) sedimentary rocks underlie the lower slopes surrounding Grand Mesa. These rocks dip gently to the northeast in the western half of the map area shown in Figure 1 and gently to the northwest in the eastern half of the map area, defining the north-trending axis of the Montrose syncline, which passes approximately through the center of the map area (Ellis & Gabaldo, 1989).

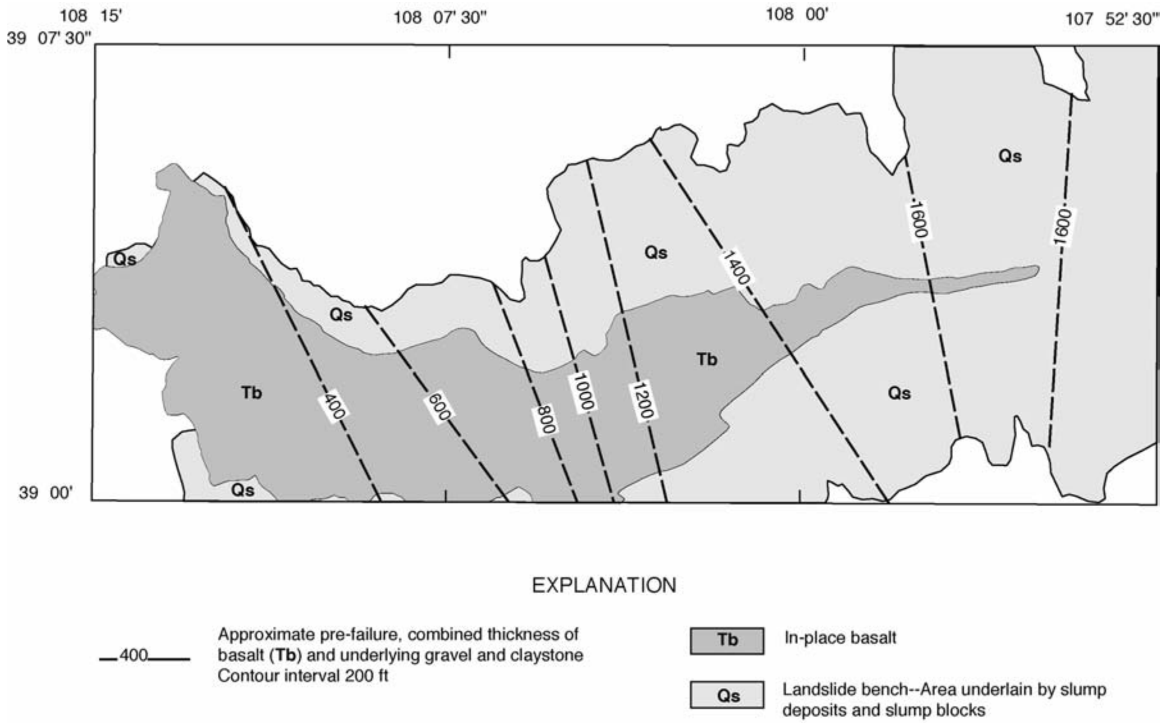


Figure 3. Approximate combined thickness of basalt and claystone involved in slumping. Area is that of Figure 1 (after Baum and Odum, 1996).

An unnamed Miocene or Oligocene (William J. Hail, jr., oral communication, 1994) unit of gravel and claystone unconformably overlies the older rocks. The unconformity appears to dip gently to the west and the gravel and claystone unit thickens from a wedge edge southeast of Lands End to about 800 ft (240 m) beneath Crag Crest (Figure 1). A thick sequence of Miocene basalt flows caps the mesa (Marvin et al., 1966). The basalt also thickens to the east but dips gently to the southwest. Slumping of the basalt and underlying claystone has destroyed much of the former basalt cap of Grand Mesa and created a broad bench that surrounds the mesa, which Yeend (1969) informally called the landslide bench. Many ridges and small hills cover the landslide bench, but on average it slopes gently (2°-5°) away from the mesa. Glacial and periglacial deposits of Pinedale (?) and Bull Lake (?) age cover much of the area, including some of the slump blocks (Yeend, 1969). These deposits are 0-10 ft (0-3 m) thick over much of the area and 10-100 ft (3-30 m) thick in moraines and between some slump blocks. Earth-flow and soil-creep deposits several meters thick cover many of the lower slopes in the western one-third of Grand Mesa (Yeend, 1969).

Most slump blocks probably moved during the Pleistocene and are presently inactive; however, a few incipient blocks (blocks that have been displaced less than a few meters) may have first moved during the late Holocene. Most blocks probably slumped to their present positions before the last glaciation of Grand Mesa (Pinedale); fresh glacial striations are present on both sides (scarp slope and back slope) of several slump blocks, and undisturbed till of Pinedale age is present in valleys between slump blocks. Had striations occurred only on the back slopes of the blocks (former mesa surface) and the till been absent or disturbed between blocks, the blocks would clearly be postglacial features (Yeend, 1969). A few incipient blocks were active in the

1960's and moved 0.17-0.60 in/yr (0.43-1.5 cm/yr); however, monitoring over a period of eight years (1963-1971) detected no movement in others (Yeend, 1969; 1973). Assuming continuous movement at these rates since their inception, and dividing the total displacement of the active blocks by their rate of movement, we estimate that some of the active blocks (Yeend, 1973, his locations 1 and 3) could have started moving as little as 130-1500 years ago. Thus even though movement rates have undoubtedly varied through time, it seems probable that the actively moving incipient blocks first moved during the Holocene. Inactive incipient blocks may have moved either during Pleistocene or Holocene time.

Seismicity

Seismicity of the Colorado Plateau province in western Colorado is low to moderate (Kirkham and Rogers, 1981) and it seems unlikely that the slumps at Grand Mesa were seismically induced. The nearest fault that can be demonstrated to have moved in Quaternary time is 30-40 km away. Keefer's (1984) threshold values for coherent slides indicate that active faults near Grand Mesa would need to produce a M6.0 to M6.8 or greater earthquake to trigger slumps. Historic earthquakes from 1870-1979 within 90 km of the study area were of magnitude 5.5 or smaller (Kirkham and Rogers, 1981). The nearest historic earthquakes were northwest of Paonia (MMI VI, Sept. 9, 1944; M 4.4, Nov. 12, 1971) and southwest of Grand Junction (M 4.0, Jan 12, 1967), 30-70 km from the study area.

Climate

Precipitation of the study area is characterized by heavy winter snowfall and moderate amounts of rain during the summer. Precipitation was measured at stations near Mesa Lakes (elev. 9800 ft/ 2987 m) from 1971 to 1979 and Bonham Reservoir (elev. 9851 ft/ 3002 m) from 1964 to at least 1994 (NOAA, 1964-1994). Records for certain years are incomplete, so long-term averages computed for these stations are lower bound estimates (Table 1). On average, about 60 percent of annual precipitation accumulates during the six months from November through April, with maximum precipitation in March. Average annual precipitation from October 1971 through September 1978 was 26.57 in. (675 mm) at Mesa Lakes, and 29.44 in. (748 mm) at Bonham Reservoir. Annual precipitation for the two stations agreed within ±2.4 in. (±60 mm) for most years except two, when precipitation at Bonham Reservoir exceeded that at Mesa Lakes by at least 10 in. (250 mm).

The precipitation produces abundant surface and ground water at Grand Mesa. Scores of lakes and reservoirs cover the mesa and surrounding landslide bench. Stream discharge was measured for several years at small drainage basins on the south side of Grand Mesa (Table 1, and USGS, 1972-1975, 1977a, 1977b, 1978-1994). Average annual surface runoff ranged from 3 in./yr (77 mm/yr) to 37.48 in./yr (952 mm/yr). Measurements at Kiser Creek (elev. 7969 ft/ 2429 m), 20 in./yr (508 mm/yr), probably give the best estimate of runoff from the landslide bench because roughly 90 percent of the drainage area is on the landslide bench and there are no diversions. A significant fraction of the Cottonwood Creek and Young's Creek basins are downslope from the landslide bench where precipitation may be less. The average runoff at Kiser Creek amounts to 60 percent of average precipitation at Bonham Reservoir (Table 1). The remainder recharges the water table, evaporates, or is transpired. Evapotranspiration has not been measured at Grand Mesa, but probably accounts for most of the remainder, leaving perhaps a few inches (tens of millimeters) per year for deep infiltration. Whitewater Creek, which flows from the base of the basalt cap at the west end of Grand Mesa, is evidently supplied by water that has percolated through the basalt from the top of the mesa (Figure 1). Other smaller streams and springs also flow from the base of the cap rock where it intersects the landslide bench. The abundance of lakes and other natural surface water on the landslide bench is consistent with a relatively high water table in the landslide deposits on the bench.

DISTRIBUTION, MORPHOLOGY, AND STRUCTURE OF SLUMP BLOCKS

Distribution

Slump blocks are widely distributed on the landslide bench. In most areas the blocks are subparallel to the mesa edge, forming straight rows where the cliff is straight and concentric patterns in semicircular re-entrants, as along West Bench (Figure 2, also Baum and Odum, 1996). Spacing is variable; blocks commonly appear to have from a quarter to several block widths between them. In the area between Horse Mountain and Eggleston Lake (Figure 1), smaller blocks appear to be perched atop larger ones. Some of these may be resistant knobs left as surrounding basalt has weathered and toppled away. A similar situation exists, on a smaller scale, in the slump blocks south of Mesa Lake. Near the downslope edges of the landslide bench, slump blocks typically have low relief and are rounded, highly weathered, and heavily wooded, which makes their identification as slump blocks less certain than elsewhere. Blocks are sparse or absent in the areas of smooth, rolling hills north of Griffith Lake (Figure 1) and surrounding Bonham Reservoir (Figure 1); these hills are underlain by deformed red claystone.

Block size evidently increases with the thickness of the material involved in the slumping. The size of individual slump blocks increases from west to east, reaching a maximum near Crag Crest (Figure 1), just as the combined thickness of the basalt and underlying gravel and claystone unit increases from west to east (Figure 3). This relation between block size and thickness is not strictly linear, because blocks become proportionately longer and wider as thickness increases.

Morphology

Incipient blocks exist at the edges of the mesa where adequate thickness of claystone exists to allow their development; we examined several examples along the north edge of the mesa. Some of these are only several feet wide and have separated from the mesa along fractures that are 15-100 ft (5-30 m) long. The fractures have opened up to 3 ft (0.9 m), and 10-20 ft (3-6 m) deep (Figure 4). Other incipient blocks are tens of meters wide and hundreds of meters long. One of these, near Crater View on the north edge of the mesa (Figure 1), is well developed and displays features characteristic of blocks in their earliest stages of movement. The 1100-ft (330-m)-wide block has been tilted sideways; it is down about 6 ft (2 m) at its east end and 0 ft (0 m) at its west end (Figure 5). A scarp bounds the block at its east end. The scarp gradually changes and decreases in height to the west, first to a trench formed by an open vertical joint that is partially filled with rubble, and then, near the west end of the block, to a series of 2-5-ft (0.5-1.5-m)-deep pits that are partially filled with soil. One of the pits opens to a small cavern formed in an open vertical joint. Inside the cavern (Figure 6), points formerly in contact on the joint surfaces are

Figure 4. Photograph, looking west, showing open tension crack on north edge of Grand Mesa.

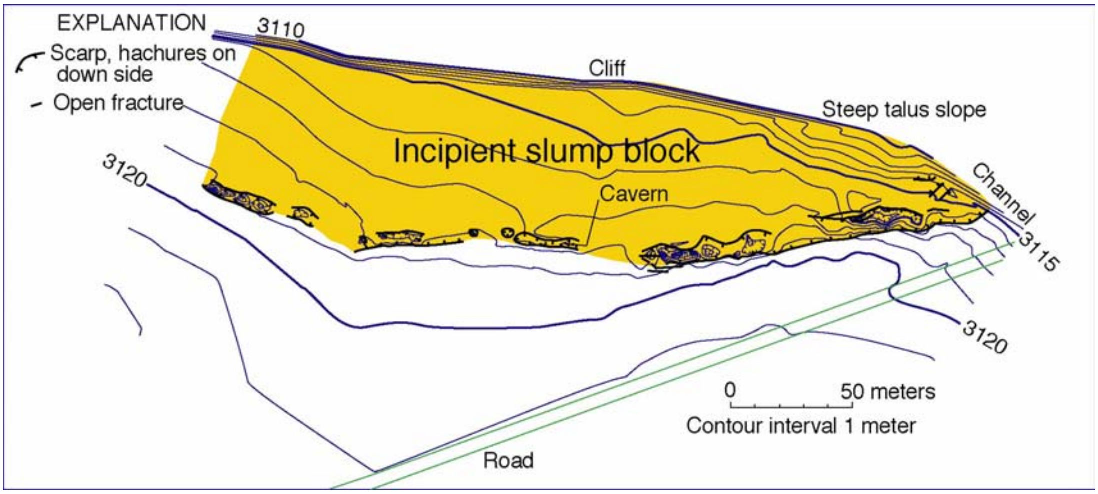
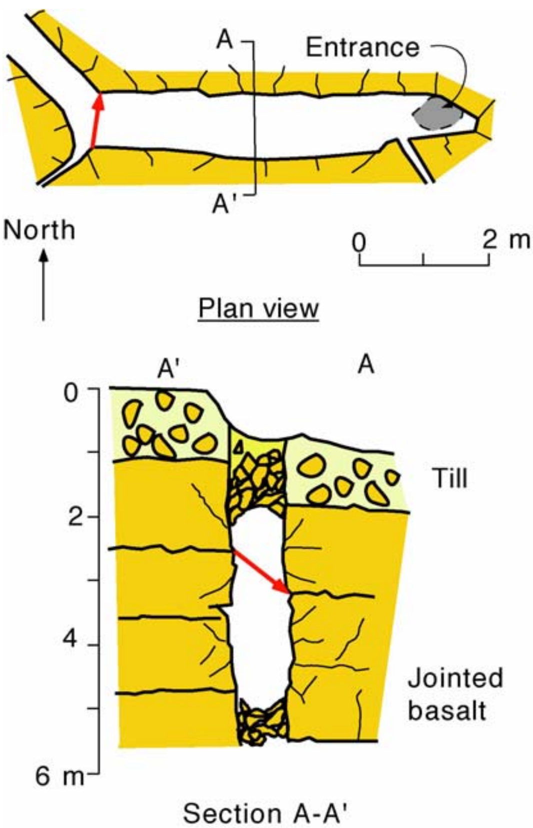
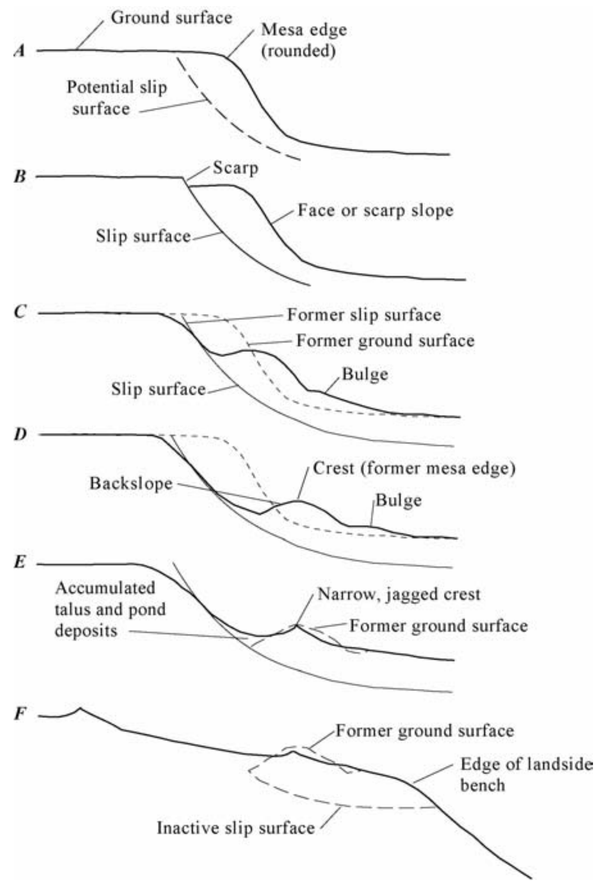


Figure 5. Fractures and other surface features of an incipient slide block near Crater View.

now about 2-2.5 ft (0.6-0.8 m) apart horizontally and have moved downward 2.5 ft (0.8 m). Despite downward (shear) displacement across the joint, the joint surfaces are undisturbed and show no crush-marks or striations indicative of shearing. The open vertical joints forming the trench, cavern, and pits, and the lack of shearing of the joint surfaces, are consistent with initial lateral movement of the slump block. Open fractures at other locations along the mesa edge provide additional evidence that the mesa cap is spreading laterally and separating into blocks (Figure 4). The downward displacement is consistent with subsequent backward rotation of the slump block. Several other slump blocks around the mesa have tilted laterally as at this location.

Slump-block profiles change gradually as movement and degradation progress. The western part of the map area has been free of glacial ice since the end of Bull-Lake (?) time and glacial processes have done little to alter or obscure profiles of the slump blocks. Study of Baum and Odum's (1996) map and cross sections, supplemented by field observation, shows that blocks have similar, though less distinct, profiles in areas glaciated during Pinedale (?) time. Figure 7 shows profiles of blocks after various amounts of movement and weathering, based mainly on observations in the western part of the area (Figure 1). The initial profile of a slump block depends on the topography of the mesa top and the underlying basalt. The mesa top undulates





Figures 6 (Left) and 7 (Right). Figure 6. Sketch map and cross section of cavern at head of incipient slump block near Crater View. Figure 7. Schematic profiles of slump block evolution (after Baum and Odum, 1996).

gently and slopes toward the southwest. Some flow units in the basalt are thick and massive and form near-vertical cliffs 20-60 m high at the mesa edge (Figure 8), whereas others ravel as an adjacent slump block subsides leaving the mesa edge rounded at the top and the scarp covered with talus (Figure 2). Raveling and talus accumulation has occurred on scarps as small as 3-12 m high (Figure 9). Thus, some slump blocks start out with a nearly flat top, sharp or slightly rounded edge, and a steep, nearly vertical, face; others start with a nearly flat or undulatory top, a rounded edge, and a sloping, talus-covered face (Figure 7A). As a block rotates, dropping away from the mesa and tilting towards it, the relict mesa surface forms a back slope and the former mesa edge becomes the crest of the slump-block ridge (Figure 7B). The back slope gradually becomes steeper as downward displacement (and backward rotation) increase (Figure 7C). This relation between increasing downward displacement and tilt is apparent in slump blocks south of Mesa Lake, northwest of Island Lake, near Cold Sore Reservoir (Baum and Odum, 1996, their section D-D'), and near West Bench (Figure 1). Meanwhile, a linear or crescent-shaped depression forms at the base of the back slope, between the block and the new mesa edge (Figure 2). Ponds or lakes, such as Island Lake (Figure 1), may occupy the depression; most lakes in the area, although retained by artificial dams, occupy such depressions (Schuster, this volume). A low bulge or ridge commonly forms downhill from the coherent slump block (Figure 7C and 7D)

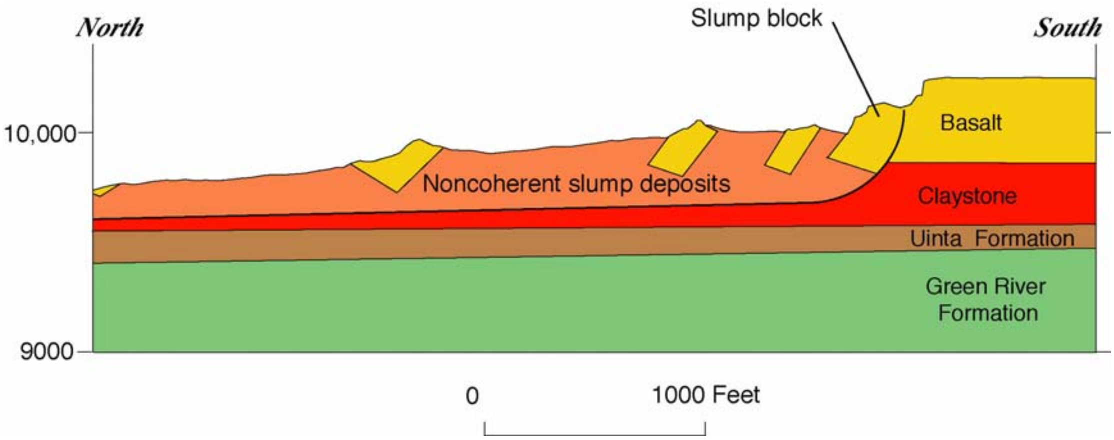


Figure 8. North-trending geologic cross section just west of Crater View, looking east (after Baum and Odum, 1996, section B-B').

Figure 9. Scarp covered with basalt talus. Scarp is 12 m high. Grassy area shown in left half of photograph is back-tilted surface of a slump block above the Mesa Lakes area.

probably as a result of compression of claystone and surficial deposits below and ahead of the slump block. Such bulges are apparent downslope from several coherent slump blocks on color aerial photographs of the Mesa Lakes and West Bench area (Fig. 2). Finally, after much downward movement and backward rotation, the block reaches the landslide bench (Figures 2, 7D).

By the time a block reaches the landslide bench, its slopes begin to flatten; the depression gradually fills with talus deposits and pond sediment until most of the former mesa top is buried (Figure 7E). Meanwhile the crest of the ridge (former edge of the mesa) erodes and ravels to form a narrow, jagged crest ridge and most of the scarp slope below becomes covered with talus deposits (Figure 8). Many blocks near the edge of the landslide bench are soil covered and forested, and have low, rounded, asymmetrical profiles. Such blocks probably represent a late stage of slump-block evolution (Figure 7F). Retreat of steep slopes below the landslide bench undermines blocks near the edge of the bench, resulting in their incremental collapse over the edge of the landslide bench.

Geometry and Structure

Most slump blocks are rectangular, crescentic, or lenticular in plan view (Figure 2); a few have irregular shapes such as the incipient block underlying Cold Sore Reservoir (Baum and Odum, 1996). Regardless of shape, the long dimension is subparallel to the mesa edge. Maximum width is 0.1-0.6 of the length. Strongly crescentic blocks commonly break into 3 or more main pieces, separated by transverse grabens that are roughly parallel to the direction of movement. The most obvious examples of broken, initially crescent-shaped blocks are west of Mesa Lake and on the West Bench (Baum and Odum, 1996).

Layering of the basalt is poorly exposed in most ridges because some flows are thick and massive; thinner flows are strongly jointed and tend to ravel and thereby obscure any layering that might be present. At every place we could observe layering, it typically dips toward the mesa (Baum and Odum, 1996). At some places on the West Bench (Figure 1), the dip direction is strongly oblique to the mesa edge, which is consistent with the observation that some blocks tilt to one side as they move downward. We found layering dipping away from the mesa in only one location, where a small basalt block that had toppled from the mesa edge onto slump blocks below. Dip appears to increase as blocks move downward; layering in blocks still in contact with the mesa generally dips less than layering in blocks that have separated from the mesa. However, attitudes of layering in blocks on the landslide bench show little evidence that dip increases significantly (more than a few degrees) after a block separates from the mesa and reaches the landslide bench. As a result of this general dip toward the mesa, most blocks have asymmetrical surface profiles; the back slope (former mesa surface) generally slopes less steeply than the scarp slope (former cliff at edge of mesa), which faces away from the mesa (Figure 8).

We found exceptions to this typical profile in the area east of Youngs Creek (south of Crag Crest, Figure 1) and at Horse Mountain (north of Crag Crest), where many blocks have flat or rounded tops and subsymmetrical surface profiles. Based on the presence of strongly tilted blocks adjacent to Crag Crest, we believe that the slump block complexes to the north and south have similar structure to that observed elsewhere on Grand Mesa. However, lack of well-exposed layering in the basalt blocks precludes ruling out other structures, such as horsts and grabens, in
these areas.

KINEMATICS

Observation of the distribution, morphology, and structure of the slump blocks leads to several inferences about the kinematics and mechanics of slump block emplacement. These inferences lead to conceptual models for initial failure, movement of slump-block complexes, and for the shape of the failure surface.

The slumps appear to have been initiated by lateral spreading of claystone beneath the weight of the overlying basalt. Many large slump blocks have separated from the mesa along preexisting vertical joints. The open vertical joints observed along the north edge of the mesa (Figures 4, 5 and 6), and the lack of shearing of the joint surfaces, observed at an incipient slump block described previously are consistent with initial opening of the fractures, prior to downward displacement. The downward displacement is consistent with both lateral spreading and subsequent backward rotation of the slump block.

Rotational retrogressive failure, rather than the more commonly reported retrogressive failure of translatory blocks (Hansen, 1965; Voight, 1973), appears to operate at Grand Mesa. Field observations and analysis of aerial photographs indicate that nearly all blocks have moved by backward rotation combined with or followed by translation. The dip of depositional layering within the blocks toward the mesa, the increasing tilt with downward displacement of blocks still in contact with the mesa edge, and the widespread asymmetrical profiles of blocks resting on the landslide bench are all consistent with rotational movement.

Rotation alone is insufficient to explain the retrogressive failure that has created the gently sloping landslide bench surrounding Grand Mesa. Once a block has rotated 30°-50°, it typically has undergone large vertical displacement but relatively little horizontal displacement. Rotation apparently ceases because tilt increases little if at all once a block reaches the landslide bench. Outward movement must continue, mainly by translation because the presence of a block at the foot of the slope interferes with rotational movement of the new block failing above and behind it. However, a new, large block can begin to fail before the preceding block has moved more than a few tens of meters downward and forward as depicted in section D-D' of Baum and Odum (1996). Several kinematic models, such as translation with extrusion, simple shear, bed-normal compression (Muir Wood, 1994), and other models can explain how older, strongly tilted blocks continue to move forward as younger blocks fail and push them from behind. Nearly all these models rely on the movement of slump blocks over a gently dipping rupture surface that extends from the edge of the mesa to the edge of the landslide bench (Figure 8).

Shape of the failure surface

Several observations constrain the general shape of the basal failure zone beneath slump block complexes at Grand Mesa. Steeply inclined fractures occur at the heads of the complexes, where slump blocks separate from the edge of the mesa. After the head fracture opens, the blocks move downward and outward and become strongly back-tilted. Block crests on the landslide bench have relatively wide spacing and the landslide bench slopes gently away from the mesa. These combined observations indicate that the block complexes have moved on listric failure zones that are steeply inclined at the head, turn sharply beneath the slump block and gradually flatten toward the toe of the landslide complex at the downslope edge of the landslide bench. Analyses of strike and dip measurements and ground-surface profiles for several slump complexes at Grand Mesa indicate that the profile of the curving part of the failure surface is more like a cycloid or hyperbola than a circle. Both cycloidal and hyperbolic slip-surface profiles allow backward rotation of a slump consistent with the amount observed in the field; whereas a circular slip-surface profile allows less than the observed amount of backward rotation.

PHYSICAL PROPERTIES OF THE CLAYSTONE

The abundance of claystone beneath the basalt flows is probably a key factor in the widespread slumping of Grand Mesa (Yeend, 1969). We examined deformed beds of claystone and clayey sand exposed in road cuts a few kilometers west and east of the town of Grand Mesa (Baum and

Odum, 1996) and tested representative samples in the laboratory. The claystone and clayey sand are uncemented or weakly cemented, contain little or no material coarser than 0.425 mm, and behave plastically when remolded (Table 2). The claystone has high plasticity and the clayey sand has low plasticity (Figure 3 of Baum and Odum, 1996). High plasticity of the claystone is consistent with the deformation and folding observed in claystone exposures (Yeend, 1969) and with the low shear strength determined by laboratory tests (Table 2 and Figure 10).

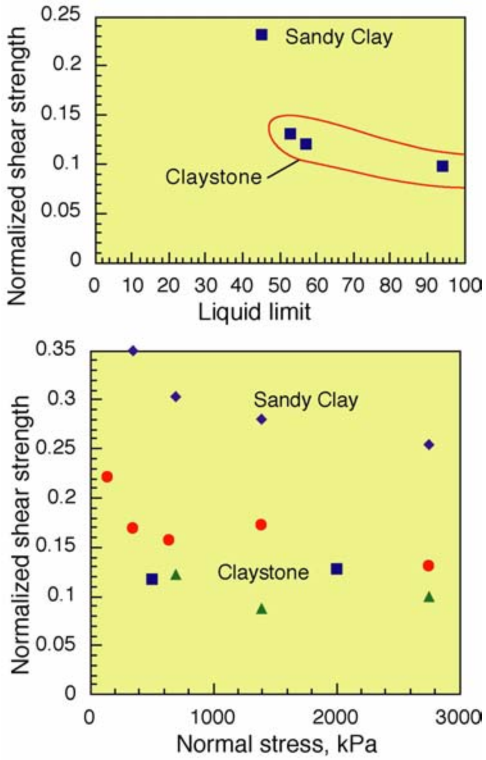


Figure 10. Relationships between normalized shear strength, liquid limit and normal stress of samples listed in Table 2. Normalized shear strength equals the shear strength divided by the normal stress.

Residual shear-strength parameters were determined by direct shear tests on remolded samples of claystone (Table 2 and Figure 10). The range of normal stresses represent values that claystone would be subject to on the landslide bench and under the basalt cap at the west end of the mesa. Normal stress under the basalt cap increases toward the east, because the thickness of the basalt increases. Expected normal stresses toward the east end of the mesa exceeded the capacity of testing equipment available for our investigation. However, Figure 10 indicates that the normalized shear strength is generally greater at low normal stress (<700 kPa) than at higher normal stress (2750 kPa). This is consistent with a decrease in the angle of residual friction as normal stress increases. Shear strength tends to decrease as liquid limit increases (Figure 10). Even the clayey sand has relatively low residual shear strength (Table 2).

CONCLUSIONS

Slump block complexes, like those at Grand Mesa, occur at many locations throughout the western U.S. and probably in many other areas of gently dipping strata throughout the world. The subparallel distribution of slump blocks on a wide, gently sloping landslide bench is consistent with movement of tandem blocks in large landslide complexes, rather than independent movement of successive, individual blocks. The relatively great width of the landslide bench compared to the drop in height from the edge of the basalt cap to the edge of the bench (or low height to length ratio of the slump-block complexes) at Grand Mesa indicates that the complexes moved on weak basal shear surfaces. Presence of a relatively brittle cap rock overlying a soft, weak mudrock or claystone appears to be essential to the initiation of the slump blocks and low shear strength of the claystone is essential to the retrogressive failure and forward movement of slump block complexes across gently sloping surfaces. The presence of open fractures parallel to the mesa edge is consistent with lateral spreading claystone beneath the weight of the basalt cap.

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Table 1. Hydrologic data for Grand Mesa, Mesa and Delta Counties Colorado.

Precipitation

Station	Bonham Reservoir	Mesa Lakes
Elevation, ft (m)	9851 (3002)	9800 (2987)
Period	Oct. 1964-Sep. 1993	Oct. 1971-Sep 1978
Mean Annual Precipitation, in. (mm)	33.27 (845)	26.57 (675)
Maximum Annual Precipitation, in. (mm)	53.07 (1348)	42.56 (1081)
Minimum Annual Precipitation, in. (mm)	15.35 (390)	16.34 (415)

Stream Discharge (Runoff)

Station	Surface Creek near Cedaredge	Kiser Creek	Cottonwood Creek	Youngs Creek
Elevation, ft (m)	8261 (2518)	7970 (2429)	7620 (2323)	7160 (2182)
Drainage area, mi^2 (km^2)	26.7 (69.2)	5.28 (13.7)	2.53 (6.6)	10.3 (26.7)
Period	Oct. 1964-Sep. 1993	Oct. 1960-Sep. 1969	Oct. 1960-Sep. 1968	Oct. 1960-Sep. 1969
Mean Annual Discharge, in. (mm)	22.40 (569)	20.00 (508)	6.54 (166)	9.25 (235)
Maximum Annual Discharge, in. (mm)	37.48 (952)	27.17 (690)	10.59 (269)	13.98 (355)
Minimum Annual Discharge, in. (mm)	5.39 (137)	13.82 (351)	3.03 (77)	5.59 (142)

[Station locations shown on Figure 1. Precipitation data from U.S. Weather Service (NOAA, 1964-1994). Stream discharges from USGS (1972-1975, 1977a, 1977b, 1978-1994). Annual precipitation has been recomputed in terms of water years for comparison with stream data. Stream discharges have been divided by drainage area to facilitate comparison with precipitation]

Table 2. Physical properties of claystone from Grand Mesa.

Sample Number	Description	Liquid Limit, percent	Plastic Limit, percent	Residual Cohesion, lb/ft^2 (kPa)	Coefficient of Residual Friction
1-1	Red claystone - - - - - - - -	53	27	0 (0)	0.13
1-2	Brown clayey sand - - - - -	44	25	--	--
2-B1	Red claystone - - - - - - - -	57	30	420 (22)	0.12
2-B2	Light-brown clayey sand -	31	24	--	--
2-C1	Green expansive claystone	61	30	--	--
2-C2	Maroon claystone - - - - - -	61	32	--	--
3-1	Gray claystone - - - - - - - -	94	45	125 (6)	0.096
4-2	Light-brown clayey sand -	45	29	1295 (62)	0.23
5-1	Light-gray claystone - - - -	65	28	--	--

[Liquid and plastic limits as percent water of total weight. Tests conducted on remolded samples. Samples tested in direct shear (2-B1, 3-1, and 4-2) were consolidated under the maximum vertical load, 57,400 lb/ft^2 (2750 kPa), before shearing. Samples were sheared through forward and reverse cycles until strength reached residual values. Samples were then unloaded and shearing was repeated to measure residual strength at successively lower normal loads. Sample 1-1 was tested in ring shear. It was consolidated under a load of 42,000 lb/ft^2 (2000kPa) and sheared through several revolutions at various speeds.]

CHEYENNE MOUNTAIN QUATERNARY DEPOSITS ORIGINS AND ENGINEERING GEOLOGY SIGNIFICANCE

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Key Terms: Cheyenne Mountain, landslide, earth flow, Colorado Springs, impacts on development

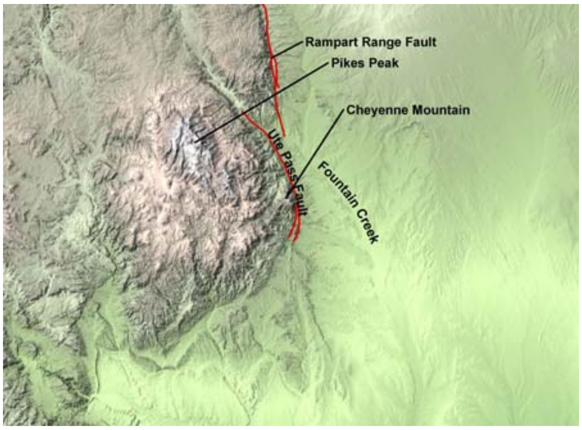
ABSTRACT

A large anomalous surficial deposit exists at the base of Cheyenne Mountain that is characterized by very large boulders of Cheyenne Mountain granodiorite, some the size of small houses. This bouldery deposit extends 1.7 mi (2.7 km) from the mountain front to Highway 115 and has puzzled geologists working in the area for many years. In earlier mapping by the U.S. Geological Survey the deposits were mapped primarily as Qrof - rock fall deposit likely of catastrophic origin (Scott and Wobus, 1973). Previously Finlay (1916) mapped the deposit as undifferentiated mesa gravels. Trimble and Machette (1979) mapped the deposit as Qls - landslide deposits. The northern portion of deposit in the Colorado Springs Quadrangle was mapped by Carroll and Crawford (2000) of the Colorado Geological Survey as Qfro - rock fall deposit and Qls - landslide with smaller areas of colluvium and younger fan deposits overlying the Qfro in some areas. The southern portion of the deposit in the Cheyenne Mountain Quadrangle has been mapped by Rowley et al. (in press) to also be Qfro - rock fall deposit and Qls - landslide with smaller areas of slope-wash colluvium, outwash gravel (Verdos Alluvium equivalent), older fan deposits, and younger fan deposits overlying the Qfro in some areas.

Recent subsurface information and larger-scale field mapping has generated additional data about this deposit and how it was formed. The Deposit is of complex origin, resulting from large earth flow deposits, rock-fall avalanche, alluvial/debris fan deposition, and subsequent modification by later landslides and alluvial-fan deposition. The majority of the earth flow deposit is likely near-contemporaneous and occurred during a significantly wetter period in the past, possibly dating to Late Pleistocene during the last glacial age. A seismic event may have triggered the failure and flowage of the earth. The deposit has partially buried earlier large landslide scarps formed from deep-seated slide surfaces in the underlying claystone of the Pierre Shale bedrock that also moved eastward, away from the mountain front. In areas where the shale bedrock is exposed and probably disturbed at and near the deposit toe, there is an increased risk of slope instability. Continual debris-flow deposition, erosion, and landsliding have modified the deposit. The complexities of the geology and continuing lack of consensus on the origin of the deposit have impacted the perceived risks for developments on and around the edges of the deposit. Recent, more extensive, investigations have led to improved understanding by all parties of the geologic conditions and possible restrictions to proposed development to prevent possible future impacts.

INTRODUCTION

Cheyenne Mountain is located in the southwestern Colorado Springs area (Figure 1). This boundary of the Front Range and the Colorado Piedmont of the High Plains province presents a steep Precambrian granodiorite mountainside that abruptly butts against Mesozoic formations as a result of high-angle reverse faulting along the Ute Pass Fault. Many theories regarding the origin and nature of a large surficial deposit (referred to as the Deposit) along the eastern flanks of Cheyenne Mountain have been circulating through the geologic community. Now, as more development is occurring and being proposed in this area, developers have commissioned geologic and geotechnical studies, and deep (greater than 50 ft [15 m]) subsurface data have become available. Concurrent with these investigations, the Colorado Geological Survey (CGS) has performed 1:24,000 scale quadrangle geologic mapping in the area, GIS and photogrammetric analyses of high-resolution ground-terrain models (White and Wait, in press), and 1:6,000 scale mapping of the Cheyenne Mountain State Park (Wait et al., in press), whose boundary includes the southern portion of this deposit. These data are beginning to provide a clearer interpretation of the Deposit.



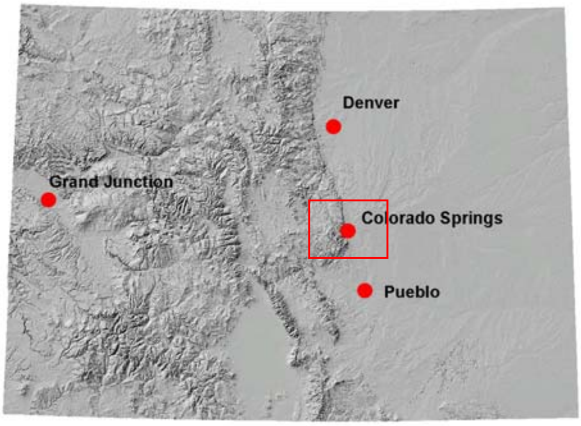


Figure 1. Colorado Location Map. Figure on left is a blowup of the Cheyenne Mountain and Colorado Springs area shown in the red box (right).

The bouldery surficial deposits along the eastern flank of Cheyenne Mountain were originally mapped by Finlay (1916) as undifferentiated mesa gravels. Later mappingby Scott and Wobus (1973) recognized that these deposits were different from typical flat-lying, mid-to-late Pleistocene pediment deposits, such as the Verdos and Slocum, and remapped these deposits as rockfall deposits, likely of catastrophic origin. Trimble and Machette (1979) mapped the area as large-scale landslide deposits. More recent geologic mapping of the area during 2000 and 2002 by the CGS (Carroll and Crawford, 2000; Rowley et al, in press) show the area as rockfall deposits over landslide deposits with smaller areas of colluvium, sheetwash, landslide, and alluvial fan deposits. Carroll and Crawford (2000) mapped Verdos Alluvium generally along the edges and below the distal margin of the deposit for the portion of that lies within the Colorado Springs Quadrangle.

Recent subsurface information and further analysis indicate that the feature is of complex origin, with an upper portion appearing to be an alluvial-fan complex. Below the upper fan areas a slope break marks a change in the morphology to a relatively flat portion of the Deposit (where alluvium is still thick). Below this flatter portion, another pronounced slope break marks a transition to a more subdued, rolling morphology, where highly weathered claystone is closer to the surface. Farther below, surficial deposits thicken substantially and well-defined earth flow morphology occurs that extends to the toe of the Deposit. The Deposit has been further modified by later small earth flow deposits, small alluvial-fan deposits, and landslides.

LIMITS OF STUDY

For the purposes of this paper, the extent of the landform described in this paper includes the central and southern portions of what was mapped by Scott and Wobus as Qrof in 1973 (Figure 2). The Deposit is centered roughly at the NORAD facility on the flank of Cheyenne Mountain near the Ute Pass Fault. The western limit of the Deposit is buried below more-recent fan deposits near the Ute Pass Fault. The northern edge of the Deposit for this discussion roughly extends from the southern edge of Star Ranch to the east west section of Broadmoor Bluffs Drive near where recent alluvium from Fishers Canyon mantles the Deposit. The Deposit spreads eastward to Colorado Highway 115 and southward to Limekiln Valley in Cheyenne Mountain State Park (currently under development at the time of this writing).

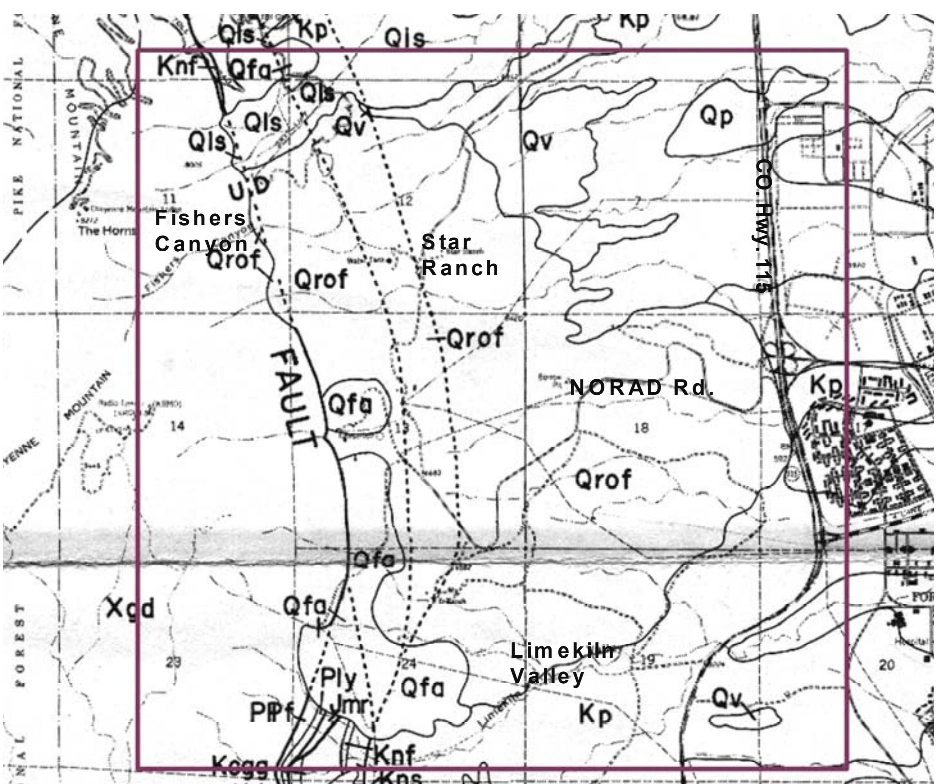


Figure 2. Scan of Cheyenne Mountain area geology from Scott and Wobus (1973). The Deposit is mapped here as Qrof, rockfall deposits. Box denotes the approximate boundary of the study area. Sections are shown and numbered on the base map used by Scott and Wobus. (Scale 1:52,000).

GEOLOGIC SETTING

The city of Colorado Springs straddles the High Plains and Southern Rocky Mountains physiographic provinces. East of Interstate 25, the city lies on rolling hills that mark the Colorado Piedmont portion of the plains. West of Interstate 25, the city rises in elevation toward Cheyenne Mountain, Pikes Peak, and the Rampart Range that border the city to the west. Within the city limits, ground elevations range from 5,720 to 9,212 ft (1743 to 2808 m) above sea level, an elevation change of 3,492 ft (1065 m) (White and Wait, in press).

Two high-angle reverse-fault zones in the Colorado Springs area, the Rampart Range and Ute Pass Faults, mark the eastern edge of the Cenozoic (Laramide Orogeny) uplifting of the Front Range. These faults have shown continued movements into the Quaternary Period (Kirkham and Rogers, 1981). Tilted, steeply-dipping, even vertical and overturned rock formations formed by uplift and by thrust-fault drag are found in the late Paleozoic and Mesozoic rock formations in the foothills of the mountain front. Younger Cretaceous and Cenozoic sediments become gradually less tilted to the east. These overconsolidated claystone and shale-rich formations, such as the Pierre Shale, dominate much of the terrain along the foothills west of Interstate 25.

Pleistocene erosion and deposition processes eroded basement rock from the Front Range, moved sediment from the mountains, and deposited sand and gravel on pediment surfaces that cap the bedrock. Late Pleistocene and Holocene erosional downcutting has incised these pediments and underlying bedrock, forming high mesas from the pediment remnants. Holocene deposition of alluvial and debris flow sediment continues to occur along the mountain front. All of these processes have combined to create the modern topography observed today.

Along the eastern flank of Cheyenne Mountain, the primary surface and subsurface geologic units that influence the Deposit are the granodiorite of Cheyenne Mountain, and the Pierre Shale. Rowley and others (in press) have placed the age of the granodiorite at 1.7 billion years, described as a medium to coarsely crystalline porphyritic biotite-granodiorite that is characterized by gneissic foliation. The Pierre Shale is a Cretaceous marine clay shale with frequent bentonitic beds (Scott and Wobus, 1973). The Pierre Shale locally weathers along a continuum from hard fissile shale to claystone to clay. Immediately south of the Deposit are exposures of resistant Fountain Formation and Lyons Sandstone and softer Lykins Formation (Wait et al., in press). Rowley et al. (in press.) show that these formations fault out along the Ute Pass fault trace concealed somewhere beneath the Deposit.

The Ute Pass Fault has an approximate length of about 44 miles (71 km) (Widmann et al, 2002). Locally, the Ute Pass fault primarily offsets Precambrian bedrock against Paleozoic and Mesozoic formations, and has created a drag effect in the underlying sedimentary beds, in some cases overturning them. According to Kirkham and Rogers (1981) “The best evidence for Quaternary fault activity is limited to the south end of the fault system near Cheyenne Mountain where development of a prominent scarp in Verdos Alluvium and scarps extending through Pleistocene rockfall deposits indicate youthful fault activity .” However, not all geologists agree on this evidence and dating of the most recent movements on the fault. We hypothesize that one of the fault scarps mapped by Scott and Wobus (1973) may, in fact, be the main landslide scarp related to the discussion of this paper.

Deposit Location

The Deposit is generally bounded on the north by alluvial fans, Verdos Alluvium, and slide deposits. In the area just north of NORAD Road and west of Colorado Highway 115, portions of the Deposit have obscured the scarp of an older landslide in Pierre Shale. The southern edge of the Deposit is also partially defined by landslide complexes on the north side of Limekiln Valley in Cheyenne Mountain State Park (Figure 2).

The Deposit is roughly fan-shaped with lobate toe features. The surficial deposits consist of an erratic mixture of large granodiorite boulders, cobbles, gravel, sand, weathered claystone, and clay. Areas in the center of the Deposit can include quite thick alluvial deposits overlying clayey landslide debris and have been covered by thin mantles of more recent slope wash. The granular materials in the surficial deposits are generally derived from the granodiorite and clayey materials from the Pierre Shale. In the southern portions of the Deposit, occasional sedimentary rocks from the Dakota and Lyons Formations have also been observed. The lobate toe of the Deposit overlies very weathered and possibly disturbed claystone. The morphology suggests earth flow processes and colluvial slope-wash deposition near the toe.

PREVIOUS INTERPRETATIONS OF THE DEPOSIT

Finlay (1916)

The earliest widely published mapping of the area was by Finlay (1916) as part of the 1:125,000-scale U.S. GEOLOGICAL SURVEY Colorado Springs Geologic Atlas Folio No. 203 shown on Figure 3. Finlay describes the mesa gravels as follows:

> "consists mainly of fragments of granite that are prevailingly angular, though the larger pieces, by their rounded surfaces, show wear and tear of stream work. They are of many sizes. The larger are 3 or 4 feet across, but most of the grains measure not more than a fraction of an inch. The deposits include a few unusually large blocks of granite, which have rolled down from the steeper mountain slopes, as from the flanks of Cheyenne Mountain. All the fragments of granite, large and small, are rudely sorted and interbedded with abundant sandy layers. Not only the granite but every other rock has made its contribution, so that the deposit includes many fragments of gneiss, schist, sandstone, and limestone, as well as quartz and pegmatite. Probably 90 per cent of the whole mass, however, is made up of Pikes Peak Granite."

Finlay included this unit with the major Pleistocene pediment alluvium units of the Colorado Piedmont, including the Rocky Flats, Verdos, and Slocum units. There was no discussion on the distinctive difference in the morphology of the Deposit.

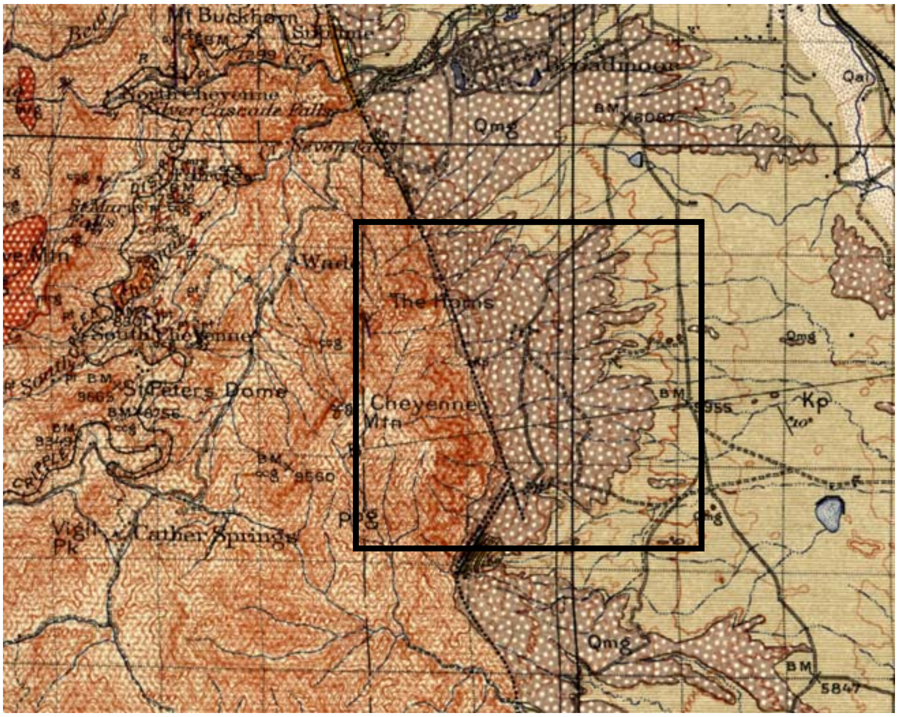


Figure 3. Cheyenne Mountain area geology from Finlay (1916). Qmg is described as being "mesa gravels from the mountain slopes." Box denotes the approximate boundary of the study area. (Scale 1:115,000)

Scott and Wobus (1973)

The next widely published map of the area was a 1:62,500-scale U.S. Geological Survey Reconnaissance Geologic Map of Colorado Springs and Vicinity by Scott and Wobus (1973), shown on Figure 2. Scott and Wobus recognized that this deposit was unlike the relatively flat, gently sloping, Pleistocene pediment surfaces in the area and first hypothesized the idea of a large rockslide or rock avalanche to explain this deposit. They mapped the Deposit as Qrof - Rockfall Deposit (Pleistocene - Yarmouth? Interglaciation). Their description of the Deposit was:

> "Extensive deposit of granodiorite boulders formed by large catastrophic rockfall off the east face of Cheyenne Mountain. Rockfalls may have been caused by an earthquake, and rocks probably moved rapidly downslope on a cushion of air. Boulders are as large as 20 feet [6 m] in diameter. Most of those below the soil line are deeply weathered. Deposit contains little fine-grained material. In eastern part of area a thin layer of granodiorite boulders overlie the Pierre Shale. Thickness probably more than 20 feet [6 m]."

They also note that subsequent earthquakes could trigger additional rockfall events. Scott and Wobus' mapping of the Deposit was reproduced on the U.S. Geological Survey Pueblo 1° x 2° geologic quadrangle by Scott et al. (1978) and the mapping by Cochran (1977) for El Paso County.

Trimble and Machette (1979)

The Geologic Map of the Colorado Springs - Castle Rock Area, Front Range Urban Corridor, by Trimble and Machette (1979) is a smaller-scale map (1:100,000) that shows the Deposit as indistinguishable from smaller areas of Qls - Landslide. Their mapping of the area is shown on Figure 4. The Qls unit is described as:

> "Includes slumps, debris flows, earth flows, rockfall avalanche deposits, and similar large masses of locally derived debris moved down slope by gravity".

For this area, the smaller-scale Trimble and Machette map was basically compiled from Scott and Wobus (1973) with no changes in landslide contacts.

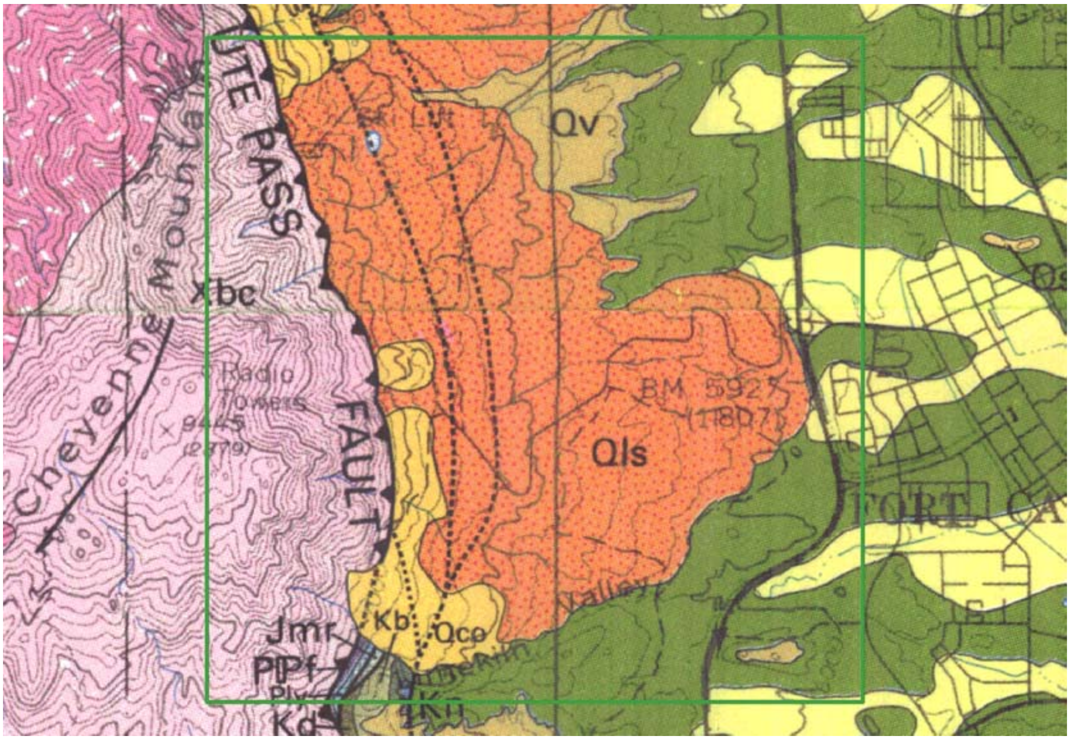


Figure 4. Cheyenne Mountain area geology from Trimble and Machette, 1979. The Deposit is mapped here as Qls, landslide deposits. Box denotes the approximate boundary of the study area. (Scale 1:28,550)

Carroll and Crawford (2000)

The CGS began a 1:24,000-scale mapping program of the Colorado Springs area in 1998. The CGS mapped the northern portion of the Deposit in the Colorado Springs Quadrangle (Carroll and Crawford, 2000) largely as Qfro and Qfro/Qls. Carroll and Crawford's mapping of the area is shown on Figure 5.

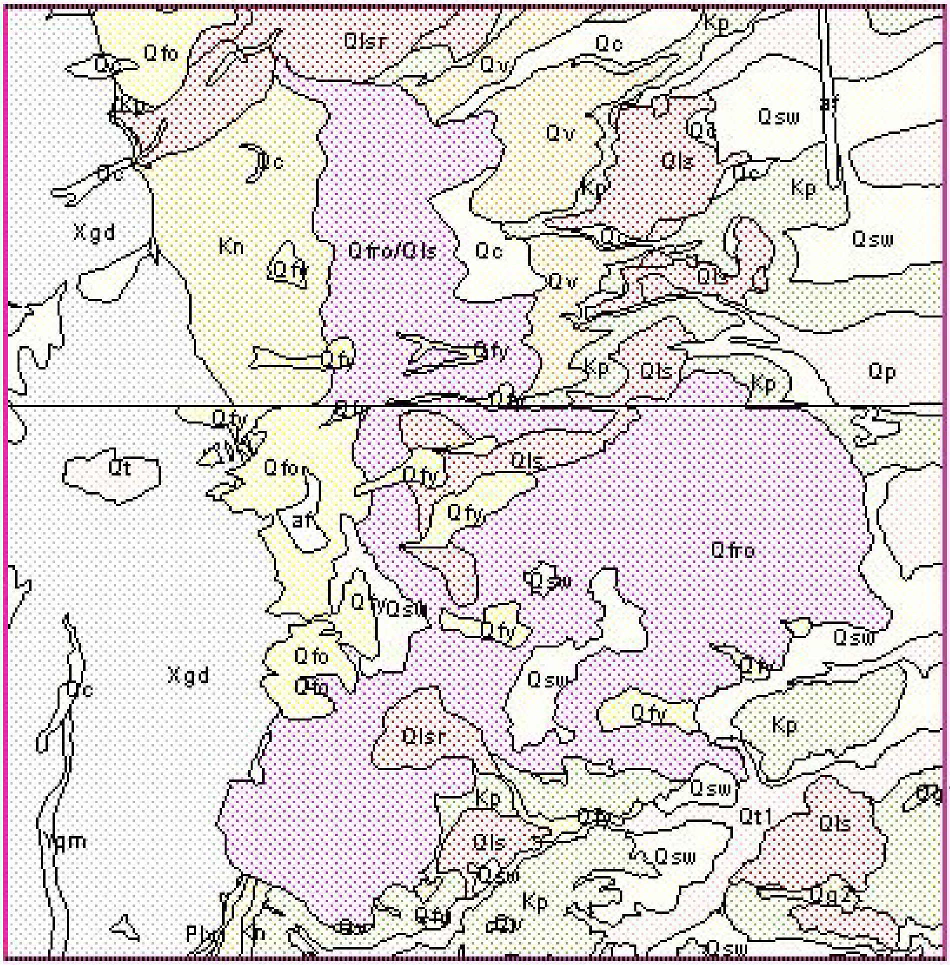


Figure 5. Cheyenne Mountain area geology as mapped by Carroll and Crawford (2000) and Rowley et al. (in press). The Deposit has been mapped here as both rockfall and landslide deposits. The line across the figure indicates where the two maps meet. (Scale 1:39,000)

Their description of the Qfro - Older fan and rockfall deposits (late Pleistocene to middle Pleistocene?) is:

> "Boulders, cobbles, pebbles, and sand in a clast-supported matrix at the base of Cheyenne Mountain. Unit is lithologically similar to older fan deposits (Qfo) near the Cheyenne Mountain Zoo but includes abundant boulders up to 25 ft. [8 m] in diameter. Unit includes rockfall, rockfall avalanche, debris flow and earth flow deposits derived from ancient slope failures in the mountains above Ute Pass Fault. Scott and Wobus (1973) originally mapped this unit as a large catastrophic rock avalanche deposit, perhaps associated with fault movement in the Quaternary period. The deposit is more likely the result of several events and consists of multiple rockfall events and other types of colluvial processes. Unit overlies both the Pierre Shale and Landslide Debris on the distal slope part (Qfro/Qls). Several flat benches within the unit at section 12, T. 15 S., R. 76 W. may reflect landslide

> scarps in the underlying deposits or Quaternary movement on splays of the Ute Pass Fault (Kirkham and Rogers, 1981)."

The Qls - Landslide deposits (Holocene and Pleistocene) are described as:

> "Highly variable deposits consisting of unsorted unstratified clay, silt, sand, gravel, and rock debris. Unit includes translational landslides, rotational landslides, earth flows, and extensive slope failure complexes with moderately to moderately well preserved geomorphic characteristics. Deposits range from slowly creeping landslides to long-inactive, middle or perhaps even early Pleistocene landslides. Landslide in the Colorado Springs region dissect the downslope side of pediment gravel two (Qg2) surfaces [Verdos Alluvium] This implies a maximum age of at least 620 ka for the larger landslide deposits. Landslides in the quadrangle all originate in the Pierre Shale or material derived from it …."

Carroll and Crawford (2000) also recognized more recent deposition on the Deposit and mapped smaller recent fan deposits (Qfy) within the larger Qfro.

Rowley et al, (in press)

Continued CGS mapping by Rowley et al. (in press) identifies the southern portion of this unit as Qfro - older landslide, fan, and rockfall deposits of late to middle Pleistocene. Rowley et al.'s preliminary mapping of the area is also shown on Figure 5. Similar to the multiple events described by Carroll and Crawford (2000), Rowley et al. theorize that the Deposit resulted from multiple processes along the Cheyenne Mountain front, and suggest the primary process as a catastrophic landslide or debris avalanche that transformed itself into a rapid earth flow at the eastern toe. They cite basement excavations in the vicinity of Ellsworth Street and Broadmoor Bluffs Drive that have shown loose breccia made up of angular pebble- to boulder-sized blocks of Pierre Shale in a clayey matrix. These displaced blocks of Pierre Shale indicate landslide movement. Rowley et al. also recognized later deposition of alluvial fans, landslides, and sheet wash that has modified the surface of this deposit.

RECENT WORK IN THE CHEYENNE MOUNTAIN AREA

Since the late 1990s, increased development along the Cheyenne Mountain front has initiated further geologic work within the Deposit. Consultants including CTL/Thompson, Entech, and Terracon, as well as mapping by CGS and more detailed larger scaled work (White and Wait, in press; and Wait et al., in press) has allowed for further understanding and interpretation of the Deposit and its geologic origins.

Recent surficial mapping by consultants working for land developers in the area has included interpretations of small landslide features (with defined head scarps and toe bulges), small fan deposits, earth flows composed of weathered claystone, boulder trains, areas of possible rafted Verdos Alluvium, and active alluvial channels within the Deposit. The boulder trains are likely the remains of older earth/debris flow side-levees from which the sand, gravel, and fines have been washed away. Similar erosion of the fines from the surface of older flows is also evident at the toe of the Deposit near Colorado Highway 115 and NORAD Road (Figure 5). In this area the ground surface is heavily mantled with 5 to 8 ft (1.5 to 2.4 m) diameter boulders.

Recently CTL/Thompson, Inc. (CTL) has drilled several relatively deep borings (80 to 150 ft, 24 to 46 m) and many shallow borings (less than 50 ft, 15 m) for geologic hazards investigations in the southern portions of the Deposit (CTL/Thompson, Inc., 2001, 2002 a, b, c, unpublished). Borings for most foundation investigations in the Deposit are generally 20 to 30 ft (6 to 9 m) deep with occasional borings to depths of 50 ft (15 m). The deeper borings were drilled to better characterize the Deposit and determine its engineering implications.

Geophysical data also were collected across a "step-and-bench" feature at the head of a lineation feature that was interpreted by Scott and Wobus (1973) as a possible trace of a fault splay of the Ute Pass Fault (Figure 6, letter a) (CTL/Thompson, Inc., 2001). Deep borings above the "step-and-bench" revealed thick deposits of alluvial fan sand and gravel. Trenching on this "step-and-bench" feature revealed a scarp and ground rotation, and formation of a wedge of organic soil on the intermediate bench. Disturbed claystone was exposed at the ground surface in another trench immediately below in a mapped seep area. Based on these trenching data, additional test pits were also excavated in suspect areas that appeared, at first, to have shallow intact claystone based on hollow-stem-auger borings and drive sampling. Examinations of trenches revealed that the claystone is highly disturbed earth flow material with scattered lenses of sand and gravel and/or more weathered zones. The thickness of these materials is usually not known, because the borings were generally terminated in the claystone under the assumption that it was a weathered zone of the underlying Pierre Shale bedrock. The displaced clay and claystone has been confirmed in basement excavations observed by authors of this paper and also mentioned by Rowley et al. (in press). Two test pits (CTL/Thompson, Inc., 2002c) in a yet-to-be-developed area within the east central portion of the deposit are shown in Figures 6 and 7. In the test pits, highly disturbed claystone was observed to have flowed plastically over sand and gravel deposits (Figure 7) or to have pushed the granular soil forward creating swirls or rolls seen in the trench exposure. In another test pit clayey colluvium appeared in contact with steep weathered and probably disturbed claystone that still displayed some bedding or layering with depth that may be an intermediate scarp. The layering may have been because of shearing in the claystone while it rafted in the earth flow. Another test pit showed evidence of past hyper-concentrated flows of sand that still occur from time to time in the area.

The recent deep borings much lower on the Deposit have encountered sand and gravel with scattered clay layers and occasional claystone layers (CTL/Thompson, Inc., 2002 a, b, c, unpublished) overlying Pierre Shale. The claystone layers in the shallower borings generally appear to be intact, however the samples are relatively small (2-inch (51 mm) diameter and 4-inches (102 mm) long) and difficult to properly interpret. The core samples from the deeper borings (deeper than 50 ft, 15 m) indicate the upper shallow claystone material is likely displaced material, because of underlying shear surfaces and occasional sand and gravel layers. The borings indicate the upper flat area is underlain by relatively thick deposits of sand, gravel, and cobbles overlying shale. The middle and lower portions of the Deposit have primarily sand and

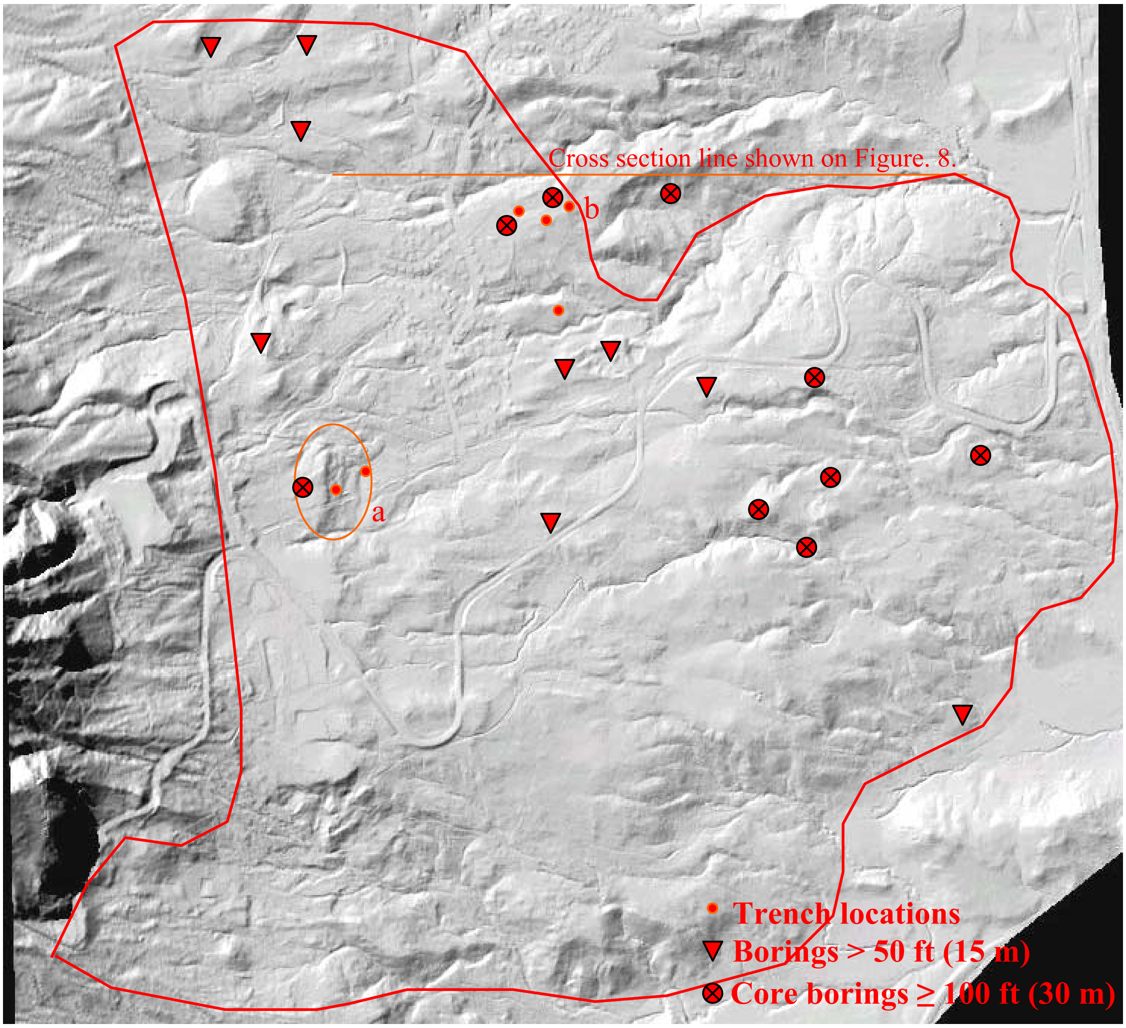


Figure 6. Shaded relief model of the study area outlined in red. Letter "a" marks step and bench area and "b" is trench shown in Figure 7. Image generated from elevation data copyrighted by Colorado Springs Utilities FIMS. Image used with permission. (Scale 1:22,000)

Figure 7. Top and bottom photos from test pit-location "b" shown in Figure 6. Note the contact of disturbed claystone and sand and gravel deposits (red lines). (Approximate scale 1:30)

gravel at the surface. The sand and gravel varies in thickness from about 20 ft (6 m) to about 105 ft (32 m) and generally thins to the east (CTL/Thompson, Inc., 2002 a, b, c, unpublished). The sand and gravel is generally underlain by weathered claystone and claystone that varies in thickness from less than 1 ft (0.3 m) to 60 ft (18 m). Some of the thicker weathered claystone areas have occasional to frequent layers of sand and gravel that indicate the material is likely earth flow. Below the step and bench feature the sand-and-gravel deposits appear to be thin, based on shallow borings that penetrated through the upper sand and gravel layers to underlying weathered claystone deposits. Trenching and borings also indicate disturbed claystone at and near the ground surface.

The contact between the lower portions of the Deposit and the intact shale usually is a thin (less than 1 ft [300 mm]) layer of weathered and distorted claystone, which appears to have been sheared (CTL/Thompson, Inc., 2002c). There are thinner shear surfaces along bedding in the shale bedrock. These bedding-surface shear zones are similar to those described by Hart (2000) and Mesri and Shahien (2003). As Hart described in his paper, there are many possible causes of bedding-surface shears including tectonic forces, lateral stress relief, and vertical stress relief, among others, that could contribute to the occurrence of bedding-surface shears in this area. The cross section shown on Figure 8 was developed across the Deposit at the location shown on Figure 6. The weak layers in the cross section are commonly bentonitic and generally are less than 1 inch (25 mm) thick, and frequently are less than 1/8 inch (3 mm).

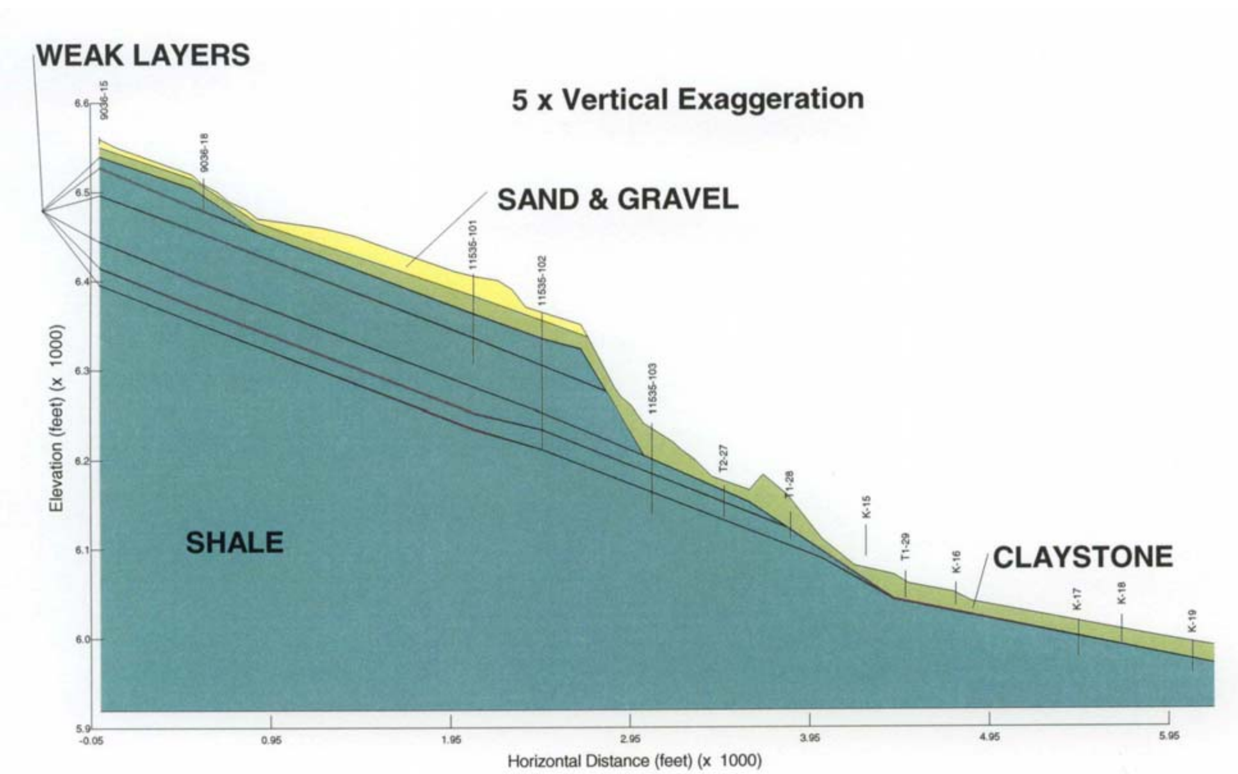


Figure 8. This cross section shown on Figure 6 has been exaggerated vertically so that the surficial deposits are visible; vertical lines indicate boring locations and depths, and the materials are generalized for clarity. The individual weak layers are shown thicker than the actual layers for visibility. This model does not include steeply dipping shale beds that may exist further west.

The eroded surface of the shale bedrock under the majority of the Deposit slopes to the east generally at an angle of about 5°. This is only a few degrees steeper than the measured dip of the bedrock in the cores of 3º-to-5º, flattening to the east. The measured residual friction angle from direct shear tests of the material along the bedding-surface shears is about 12 º (CTL/Thompson, Inc., 2002c). Stability analysis indicates that the natural slopes are stable under current climatic and geologic conditions, with the steeper slopes being marginally unstable, having factors of safety near one. City of Colorado Springs engineering staff and the local geotechnical practice generally consider 1.5 the minimum desired factor of safety for long-term slope stability for development.

Parametric slope stability analyses indicate that the ground water levels within the sand and gravel materials above bedrock do not have a large influence on the stability of the slopes. Generally the slopes that have factors of safety of at least 1.5 become marginally unstable (factors of safety near 1) under pseudo-static earthquake loading conditions (CTL/Thompson, Inc., 2002c). The pseudo-static analysis utilized horizontal accelerations of 0.08g based on values for the area from the U.S. Geological Survey seismic hazard web page (2003) for peak ground accelerations with a 2 percent probability of exceedence in 50 years. The pseudo-static analyses indicate that seismic forces will cause large blocks to fail along bedding-surface shears at the top of and within the Pierre Shale. This scenario is supported by field and borehole observations showing what appear to be blocks of disturbed claystone moving along bedding surfaces.

THE DEPOSIT CHARACTERISTICS AND MORPHOLOGY

The authors' characterization of the Deposit most closely approximates the description in Rowley, et al (in press) with some differences. Based on the subsurface work and field investigations, the following history and description of the Deposit is proposed:

The formation of the Deposit was initiated by a large-scale translational or block slide (or series of slides) along bedding surface shears that occurred under wetter conditions than at present, possibly triggered by seismic events. These wet conditions allowed the feature to mobilize as an earth flow in the lower toe area. The labeled morphology of the Deposit is shown on Figure 9. The numbered areas on Figure 9 refer to areas within the Deposit, and the lettered areas refer to geologic events that have modified the deposit morphology or were obscured by the Deposit.

Our interpretation of the formation and later modification of the Deposit follows and references the numbered and lettered area and features on Figure 9. We have attempted to follow Varnes (1978) descriptive methodology. The Precambrian granitic rocks on the west side of Ute Pass Fault (curved line along the left side of Figure 9) were uplifted, with associated folding of the sedimentary bedrock. The surface of the sedimentary bedrock was eroded to a relatively uniform eastward sloping surface, with Pierre Shale exposed. Glacial outwash gravel pediments and terraces covered the surface (E areas on Figure 9). The most extensive of these outwash deposits was the Verdos Alluvium. Closer to the granitic rocks of Cheyenne Mountain a wedge of coalescing alluvial fans was deposited. A remnant of these coalescing alluvial fans is area 4

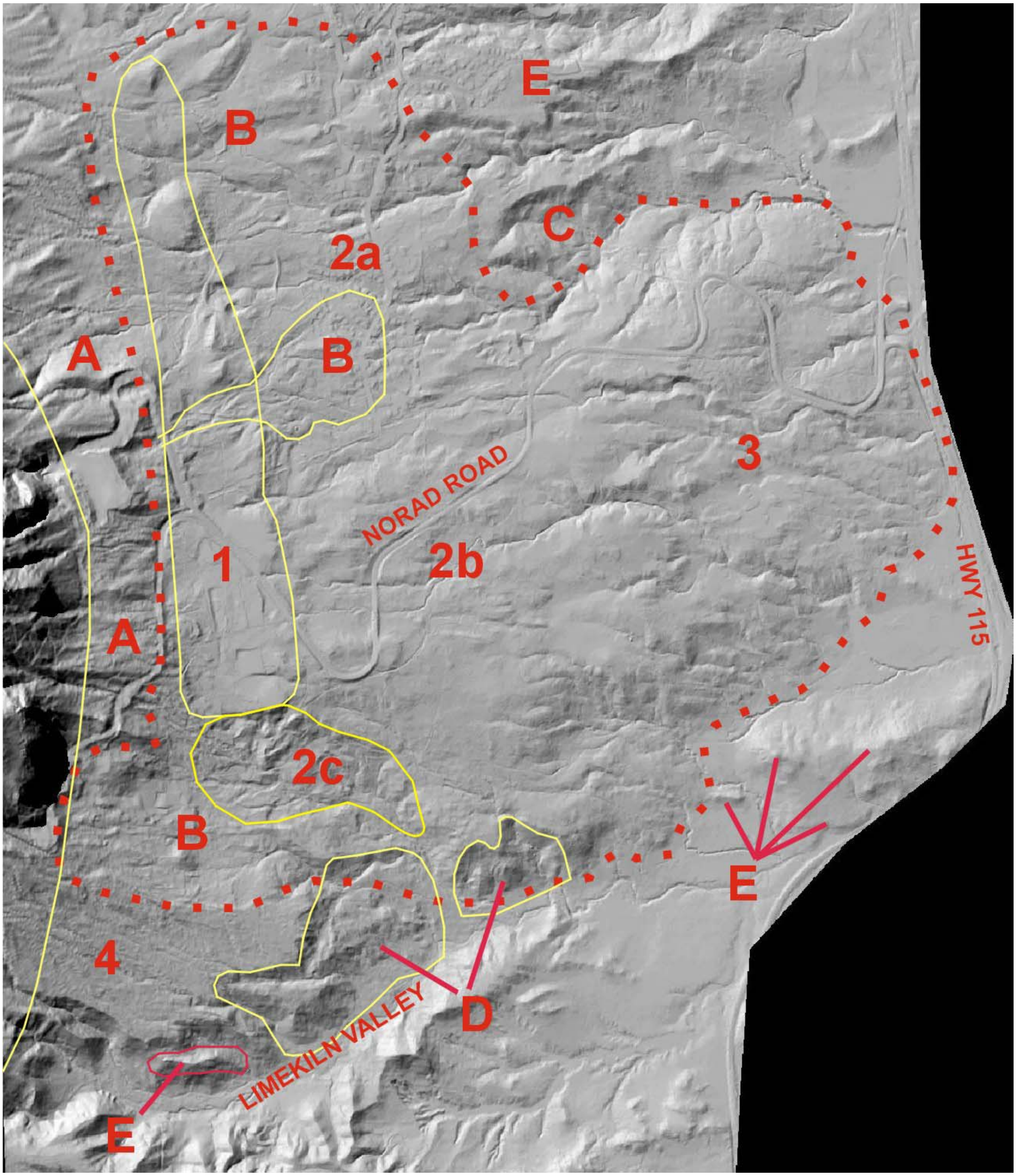


Figure 9. The Deposit and related features. Dashed red line shows approximate boundary of the Deposit. Letter and number annotations show the following: 1 – Slump block, 2 – Zone of Depletion, 3 – Earth flow, 4 - Remnant fan; A – Mountain-front fans obscuring upper scarp, B – Recent debris flows, post deposit in age, C – Early landslide scarp partially covered by earth flow (3), D – More recent landslide on the edges of the Deposit, and E – Pediment and terrace remnants. (Scale 1:22,000)

on Figure 9. As the coalescing alluvial fans were building up a series of landslides and erosion of the landslide debris was creating valleys along the eastern edge of the glacial outwash pediments. A remnant of one of these valleys is Area C on Figure 9. These slide areas are still intermittently active.

During an extended wet period (precipitation levels likely at least twice current average levels) in the past an earthquake or possibly an extreme precipitation event occurred and triggered a translational slide along one or more weak bedding surfaces within the Pierre Shale. The western portion of the slide mass stayed relatively intact and rotated slightly leaving the large slump block area 1 on Figure 9. As the slide material moved downhill to the east the lower eastern portion change into an earth flow leaving the zone of evacuation (area 2 on Figure 9). The flow stopped and was deposited in area 3. The flow stopped along its southeastern edge where it abutted glacial terrace remnants. Along its northeastern edge the flow moved over an existing landslide scarp at area C on Figure 9. The Deposit has been modified since the primary event or events that led to the deposit by more recent alluvial fans (area A west of area 1 on Figure 9), a debris flow fan cutting across area 1 and being deposited in area 2 (area B on Figure 9), and complex landslides (area D on Figure 9).

The upper scarp for the slump block (area 1) has been obscured by the coalescing of more recent alluvial-fan deposits from the mountain front (labeled area A) and may correspond with the western Ute Pass Fault splay of Scott and Wobus. The alluvial fans consist primarily of an erratic mixture of silt- to boulder-size material that was shed from the face of Cheyenne Mountain as rock fall, debris flows, and hyper-concentrated flows. Recent examination of the mountain front revealed that many of these deposits are the result of shallow slumping failures of the steep mountain flank that mobilized into debris flows. This area is still considered active and exposed to debris flow hazards. Areas shown as (B) are sites of recent alluviation from debris-flows from the mountain front.

Area 2 (on Figure 9) has been split into three zones based on morphology and thickness of surficial deposits. Sub-area 2a has a much more subdued morphology with weathered and/or disturbed claystone at the surface or shallow depths with thin mantles of boulders and other granular material. Sub-area 2b has a topography that suggests some flow characteristics with many boulders and significant alluvial reworking. Sub-area 2c is located in the area where the zone of depletion wraps around the south end of area 1. Sub-area 2c has shale exposed, minimal boulders are present, and there is evidence of recent landslide activity. The western edge sub-areas 2a and 2b of the depletion zone are marked by the previously mentioned step-and-bench feature and which is located roughly at the eastern fault splay mapped by Scott and Wobus (1973). While possibly an old fault, the fault splay appears to be the remnants of a scarp left by the failure and mobilization of the earth flow portion of the Deposit. The depletion zone, where the earth flows occurred from, has disturbed and weathered claystone at or near surface. The surficial material in the depletion zone consists largely of weathered and disturbed claystone except at sub-area 2b, where the earth flow deposits are thicker, and where there is minor and thin deposition by later debris flows and colluvial slope wash from erosion of the scarp. Ground water in this area is relatively shallow (less than 20 feet [6m]) and seasonal seep areas have been mapped at the surface.

As translational movement progressed eastward the earth flow deposit (area 3 on Figure 9) incorporated the alluvial fan material, the underlying weathered Pierre Shale, and remnants of Verdos Alluvium. The lower earth flow part of the Deposit is approximately 2.5 mi^2 (6.5 km^2) in area and consists of an erratic mixture of clay, silt, sand, gravel, boulders, and larger blocks of displaced alluvium and bedrock materials. The earth flow deposits here include the lower portion of the debris-fan deposits, the underlying Pierre Shale, and what appear to be rafted portions of the Verdos Alluvium that lay in the path of the eastward flow.

Area 4 appears to be a large remnant of an original fan deposit that was not incorporated into the Deposit. This steep fan of house size boulders likely includes rockfall avalanches, alluvial, debris flow, and earth flow deposits. The elevation of this fan is higher in elevation than a remnant of Verdos pediment gravel (E) that is shown on Figure 9 (bottom and top) but we believe this fan is younger. Subsequent small-scale landslides have occurred along the distal edges of the existing fan (area 4) and the Deposit where current stream activity in Limekiln Valley has oversteepened the valley wall and undercut the surficial deposits. The base of the valley wall exposes very weathered and possibly disturbed claystone.

The deposit displaces the Verdos Pediment and thus post-dates the pediment. The southeast edge of the earth flow portion of the Deposit abuts against hills of later pediment remnants (ages Slocum and younger?) and thus post-dates these pediments also (area E right edge of Figure 9). Hyper-concentrated flows, channel erosion, small earth flows, debris flows, and slides have modified certain portions of the Deposit since the initial large landslide/earth flow.

IMPACTS ON DEVELOPMENT

The impacts of this interpretation of the Deposit as a translational slide/earth flow on future development will include, but not be limited to, the following:

- recognition of the risk of increased erosion;
- recognition of debris flow hazards;
- increased recognition of risk of future landslides and earth movements;
- post development water related issues;
- disclosure to homeowners/buyers of the potential landslide hazard.

The interpretation of the Deposit as a translational slide/earth flow indicates that the materials in the deposit are likely to be more susceptible to water erosion than other deposits. Increased flow volumes, concentrated water flow (from storm drains) or length of time water flows could lead to increased erosion. Well-entrenched gullies exist in the deposit where concentrated flows of runoff from the NORAD facility have occurred. The Deposit moved boulders long distances, and also moved large blocks of claystone, either of which can be encountered during foundation investigation or during construction through out the Deposit. Claystone, where it occurs in this area, should not be considered bedrock for foundation or engineering purposes.

The upper portion of the Deposit is interpreted as being alluvial-fan deposits, which implies that debris flows and earth flows derived from shallow sloughing failures of the mountain flank have occurred in this area in the past and are likely to occur in the future. The upper parts of the

Deposit were impacted by significant debris flows as recently as 1965 (CTL/Thompson, Inc., 2002 a, b, c, unpublished)(indicated on Figure 7). The risk of debris flows within the upper portion of the Deposit is fairly high and is being addressed by the development community at the urging of their consultants, the City of Colorado Springs, and the CGS.

Some of the clay materials have experienced shearing in the past, due to the translational movement of the Deposit. The past shearing would likely have occurred at residual shear strength levels. Slope stability analysis of the interface materials between the Deposit and the underlying Pierre Shale should be analyzed using residual shear strength for the interface and bedding shears, and fully-softened shear strength for other clayey materials.

The event that created the Deposit was likely triggered by an earthquake and/or high ground-water levels. Some of the slopes at the edges of the Deposit have been analyzed for slope stability utilizing U.S. Geological Survey probabilistic peak ground accelerations (U.S. Geological Survey, 2003) for the pseudo-static analysis to establish development limits. If earthquakes larger than those used by the U.S. Geological Survey to establish peak ground accelerations were to occur concurrent with very wet periods, future large earth flows or other mass movements would be likely.

There are many methods currently available to reduce the risk of future slide and earth flow events. However, significant earthwork is not allowed by city code in this area, because it is part of the City's Hillside Overlay Zone (Colorado Springs Planning Department, 2002). Currently, notes are being added to plats for new development in this area regarding the landslide hazard.

Another issue that relates to both the erosion and slope stability impacts are changes in the local water regime caused by development. Generally runoff increases as development occurs and more areas are made impermeable to infiltration. Detention ponds normally collect and mitigate the extra runoff. The other impact is the increase in infiltration into the ground water because of irrigation. The increased infiltration can lead to elevated ground water levels, perched ground water conditions along clay zones, increased pore-water pressures, and increased risk of slope stability problems. The increase in ground water levels can be partially controlled by under-drains installed below the sewer lines that can capture ground water and discharge it to surface drainages. In some areas irrigation has been restricted to further reduce the risk of rising ground water levels.

The recognition of these and other hazards will likely lead to an increase in the amount of open space in new developments in the area. The land that is being developed as Cheyenne Mountain State Park is an example of a developer following the recommendations of his consultants and selling a parcel with numerous geologic hazards on the southern edge of the Deposit for dedicated open space. The use of areas with high-risk geologic hazards as open space makes them assets for the community rather than a liability for the developer.

SUGGESTED FUTURE INVESTIGATIONS

It would be beneficial to have additional deep subsurface data, including the depth to bedrock, dip of the bedding within the bedrock, and saturation levels of the bedrock materials along the western edge of the Deposit close to Cheyenne Mountain. Himmelreich and Noe (1999) indicated steeply-dipping bedrock in the upper portions of the Deposit, though it has not been identified in borings. A listric fold of possibly sheared bedding surfaces in the Pierre Shale from the steeply-dipping beds mapped by Himmelreich and Noe, to the gently dipping beds identified in the core in the central and eastern parts of the deposit was not modeled for this study. Stability of the deposit may be affected by where this listric fold occurs and its geometry. Similar subsurface information concerning claystone bedding shears in the vicinity of the Verdos Alluvium northeast of the Deposit would also be useful for comparison.

Another avenue of study not completed by the authors is a profile and volumetric analysis of the Deposit and hypothesized rotation of the upper slump block (area 1 on Figure 9). Volumetric and profile comparison of the intact fan in the southern part of the area 4 on Figure 9 to the flat surface of the upper slump block may yield interesting data.

A comprehensive study of the Ute Pass Fault to assess its seismic capability and earthquake recurrence intervals is warranted considering that urban development has already occurred and continues to occur on and adjacent to the fault. There are currently hundreds of millions of dollars worth of development within less than 0.6 mi (1 km) of the fault, and the fault's capability and seismic recurrence interval are not known with any degree of certainty.

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GEOMORPHOMETRY OF SMALL DEBRIS-FLOW DRAINAGE BASINS ALONG INTERSTATE 70, EAGLE COUNTY, COLORADO

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Key Terms: debris flow, alluvial fans, drainage-basin morphometry

ABSTRACT

Debris flows originating from small drainage basins along the mountainous part of the Interstate 70 (I-70) corridor are significant hazards that periodically disrupt transportation. Human modification of many of the fans below these basins prevents the detailed stratigraphic investigation of their depositional history that is needed to determine debris-flow frequency and magnitude. An approach based on using easily-acquired topographic information to quantitatively analyze the morphology of the drainage basins above the fans may lead to useful magnitude/frequency estimates. Survey data collected using geodetic Global Positioning Systems techniques and elevation information from a Digital Elevation Model were combined to investigate the morphometric relations between 15 drainage basins and fans along I-70 in Eagle County, Colorado. Analysis showed little correlation between basin ruggedness (basin relief normalized by the square root of basin area) and fan gradient, and an apparent inverse correlation between basin ruggedness and fan area. The inverse correlation between basin ruggedness and fan area suggests that basin morphology may be useful in estimating debris-flow frequency/magnitude on fans built primarily by debris-flow processes. However, the lack of correlation between fan gradient and basin ruggedness suggest that the erodible lithology may have an influence on the morphology of fans.

INTRODUCTION

In the summer of 1999 and again in 2000 (Denver Post, 2000) debris flows originating from several small tributary basins on the north side of the Eagle River valley near Gypsum, Colorado, impacted Interstate 70 (I-70) disrupting traffic. The basins are underlain by erodible Middle Pennsylvanian evaporites, sandstones, siltstones, limestones, and shales. The drainage basins are near the western margin of the Eagle collapse center (Scott et al., 1998, Scott et al., 1999, Lidke et al., 2002). This area was subjected to widespread crustal deformation due to the flow and dissolution of evaporite in the late Cenozoic (Kirkham & Scott, 2002).

Geomorphometry or landform morphometry is the quantitative analysis of landform morphology, and in this case study we use digital elevation and survey data to describe the drainage basins and their associated fans. Fourteen fan surfaces were surveyed using Global

Positioning System (GPS) methods. Drainage basin heights and areas were determined from a Digital Elevation Model (DEM) and from topographic maps. Basin ruggedness was characterized using Melton's number (N_m), which is terrain relief normalized by the square root of basin area (Melton, 1965). Melton's number has been used to categorize basins by their dominant sediment transport process (e.g. Jackson et al., 1987, Coe et al., 1998). Sedimentary sequences of fans below basins with relatively large Melton's numbers are typically dominated by debris-flow events. Fans below basins with relatively small Melton's numbers are typically dominated by fluvial or flooding events.

To assess the debris-flow hazard on fans, information on debris-flow frequency and magnitude is needed. Many of the fans along I-70 have been removed or modified by highway and other construction activities that prevent investigation of fan stratigraphy and depositional history. The purpose of this case study is to investigate the relations between drainage basin morphometry and fan geometry to evaluate their use in debris-flow hazard assessment. In what follows we describe the physiographic and geologic setting of the study area, and outline the methods used to determine the terrain and fan morphometric variables. Our results show a lack of correlation between basin ruggedness and fan gradient and an apparent inverse correlation between basin ruggedness and fan area. We then discuss the possible influence of lithology and sediment transport processes on the morphology of these active fans and the implications for debris-flow hazard assessment.

PHYSIOGRAPHIC AND GEOLOGIC SETTING

The study area is located north of the I-70 corridor between the Dotsero and Gypusm exits in Eagle County Colorado (Figure 1). Along this section of the corridor about 70 small drainages spill debris onto the alluvial valley of the Eagle River. The study area ranges in elevation from about 6200 ft (1900 m) near the fans to about 8200 ft (2500 m) at the top of the highest drainage basins. The drainage basins range in area from about 5980 yds^2 (5000 m^2) to more than 7.2 x 10^5 yds^2 (6 x 10^5 m^2). The larger basins drain sufficient area to have ephemeral stream channels (Figure 1). The smallest basins are essentially large gullies or debris chutes (Figure 2). Fans have formed at the mouths of both the larger ephemeral drainages and the smaller basins.

The study area is located in the ~950 mi^2 (~2500 km^2) Eagle Collapse Center where Middle Pennsylvanian and Lower Permian and younger sedimentary rocks have been deformed by the migration of the underlying Pennsylvanian Eagle Valley Evaporite, a pale gray to white gypsum, anhydrite, and halite interbedded with gypsiferous siltstones, shales, sandstones, and fossiliferous limestones (Widmann, 1997, Kirkham & Scott, 2002, Lidke et al., 2002). The area was a low-relief erosion surface prior to between 25 Ma BP and 10 Ma BP when river incision triggered widespread evaporite tectonism (Kirkham & Scott, 2002). Differential unloading of evaporite under river valleys and uplands promoted evaporite migration toward the Eagle River valley deforming the younger Eagle Valley Formation, a gray to red, bedded sandstone, and the Maroon Formation, a reddish brown arkosic sandstone (Kirkham & Scott, 2002, Lidke et al., 2002). The Eagle River anticline, just south and east of the study area drainage basins, was

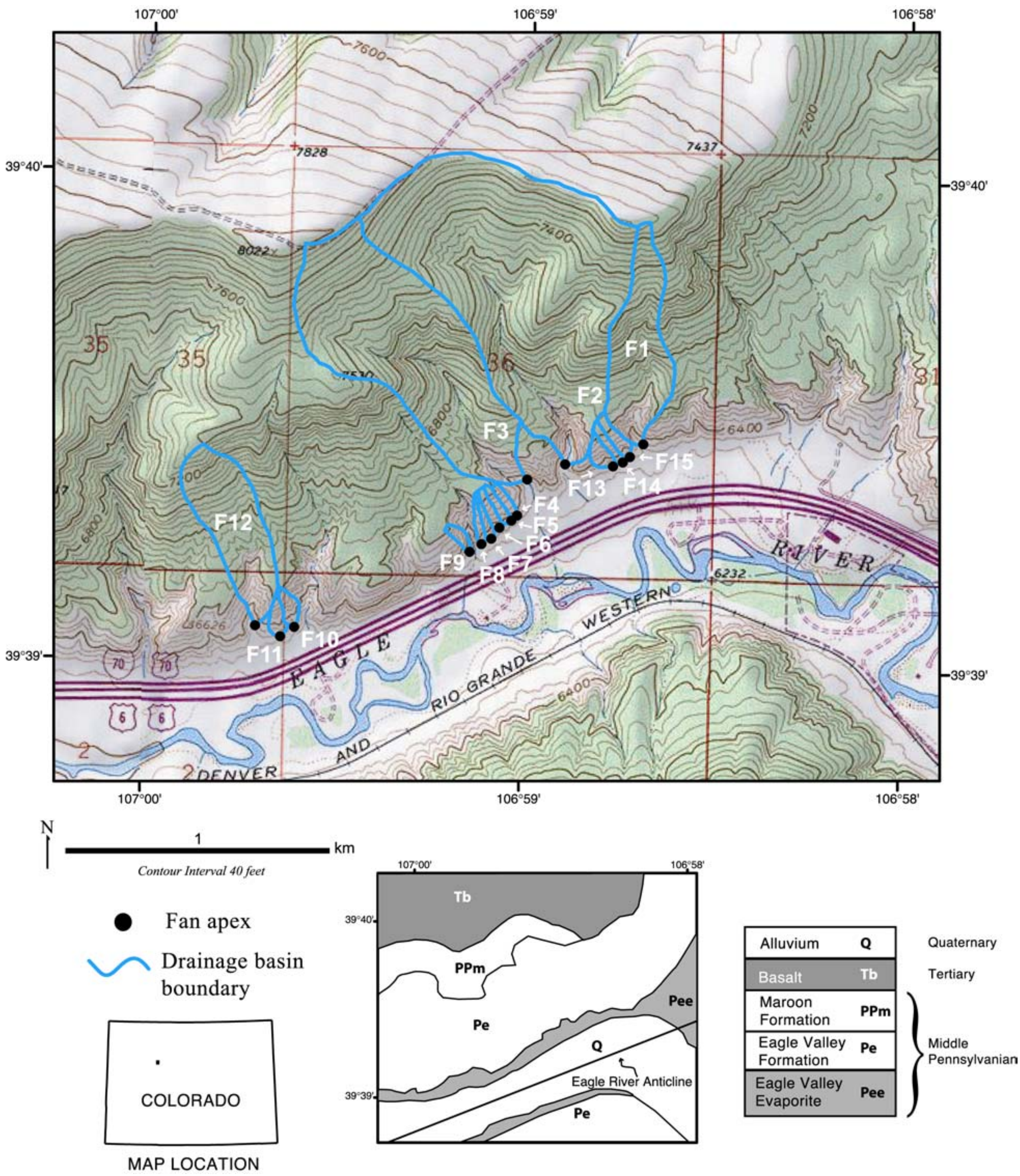

Figure 1. Map showing the location of the basins and fans in the study area. Generalized geologic map after M.R. Hudson and D.M. Moore, 2001, U.S. Geological Survey, unpublished mapping. Inset shows the general stratigraphic relation in the area (Kirkham & Scott, 2002, Lidke et al., 2002).

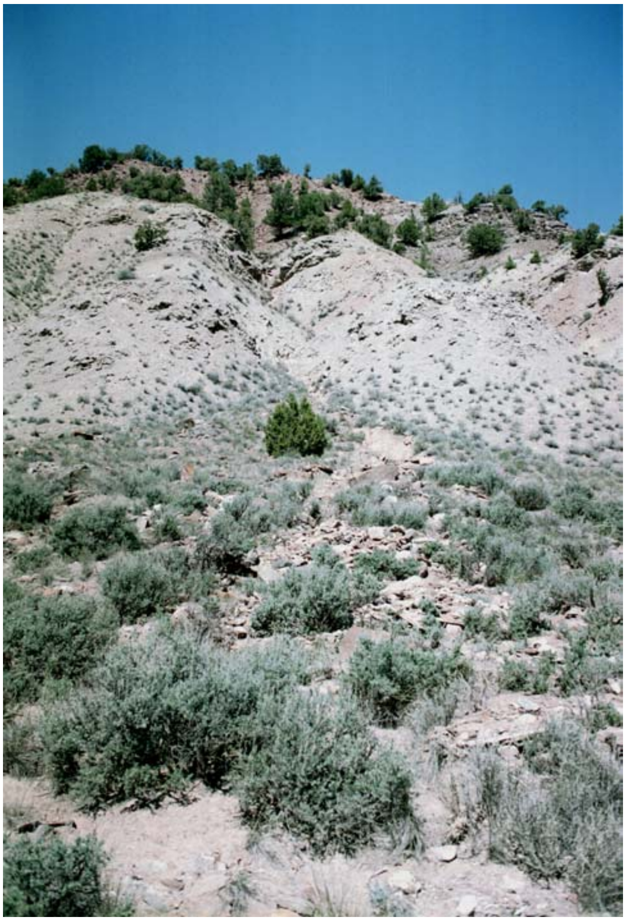

Figure 2. Photograph of drainage basin and debris fan F5 (Table 1, Figure 1). The debris fan can be distinguished from hillslopes by the presence of vegetation. Fresh debris is visible on the fan surface in the foreground. Photograph taken 4/12/00.

created by the diapiric flow of evaporite (Widmann, 1997, Lidke et al., 2002) and is expressed in the dipping beds of the overlying formations (Figure 3). In contrast to the broadly deformed overlying sedimentary rocks, the Eagle Valley Evaporite is complexly folded and subject to a variety of phenomena including hydrocompaction, subsidence, landslides, debris flows and rockfalls (Widmann, 1997, Mock, 2002). Hillslopes in the study area are mantled with a thin (generally less than 3.3 yds (3 m) thick) colluvium composed of angular clasts of gravel to boulder size supported in a sandy silt or low plasticity sandy clay matrix (Mock, 2002).

In general, the area is best described as a high semi-desert. The vegetation cover is largely sagebrush and juniper in the alluvial valley and widely-spaced (several yards (meters)) coniferous trees at higher elevations. The mean annual precipitation is 11 in (270 mm) (measured at the Eagle FAA airport 5 mi (8 km) to the east of the study site for the period 1948 to 1994) that is roughly evenly distributed over the year with a small late summer maximum.

Debris Flows

During the late summer of 1999 several mountainous areas of Colorado were affected by debris flows, rock fall, and landslides as a result of a prolonged flow of moisture from the south related to the North American Monsoon (Adams & Comrie, 1997, Soule, 1999, Coe et al., 2002, Coe et al., 2003, Godt & Coe, 2003). Along the 3.7 mi (6 km) stretch of I-70 between Dotsero and Gypsum about 20 recent debris-flow deposits were identified in April of 2000 (Figure 3).

Figure 3. Photograph of the study area showing fresh debris-flow deposits indicated by small arrows. Interstate 70 roughly parallels the axis of the Eagle River anticline (Figure 1) and is shown in the steeply dipping beds of the Eagle Valley formation in the foreground on the left (Lidke et al, 2002). The Eagle Valley Evaporite underlies the Eagle Valley formation at the base of the hillslopes (Figure 1). Photograph was taken on 4/17/00 from the right flank of basin F3 looking east.

The presence and condition of vegetation evidenced the recency of deposition on fan surfaces. Fresh deposits were identified by the absence of small vegetation such as grasses and damage to larger plants, such as the stripping of leaves, indicated the recent passage of coarse debris (Figure 4). However, new deposition did not cover the entire fan surface often leaving some of the vegetation intact.

Figure 4. Photograph of the surface of fan F1 showing damaged vegetation, large boulder (~1.5 m in diameter), and newly incised gully (~3 m deep). Photograph taken on 4/12/00.

Recent debris-flow deposits were composed of angular, fresh, unstratified, poorly-sorted sediments (Figure 4) ranging in size from silt to boulders as large as 8.2 ft (2.5 m) in long dimension. No obvious landslide scars were observed on hillslopes in the basins in the study area indicating that the debris flows were likely generated by the mobilization of material from hillslopes and stream channels by overland flow of water (Johnson & Rodine, 1984, Coe et al., 2002, Godt & Coe, 2003). The transport of loose colluvium by overland flow concentrated into rills and gullies on hillslopes and in channels has been identified as a process that may generate debris flows in basins that are sparsely vegetated or have been recently burned by wildfire (e.g. Johnson & Rodine, 1984, Cannon et al., 2001, Coe et al., 2002).

METHODS

Global Positioning System (GPS) techniques were used to survey 15 fans on which fresh material was deposited. Fan apices were identified in the field based on the abrupt change from the erosional surface of the drainage basins, to the depositional surface of the fans. The fan apex is spatially coincident with the drainage basin mouth (Figure 5). The location of drainage basin mouth/fan apex for a given basin may cover an area of a few square meters in the field. Additionally, many of the larger fans were incised up to 9.8 ft (3.0 m) near the apex of the fan (Figure 4). The fan apex point was surveyed on the fan surface, not in the newly incised channel.

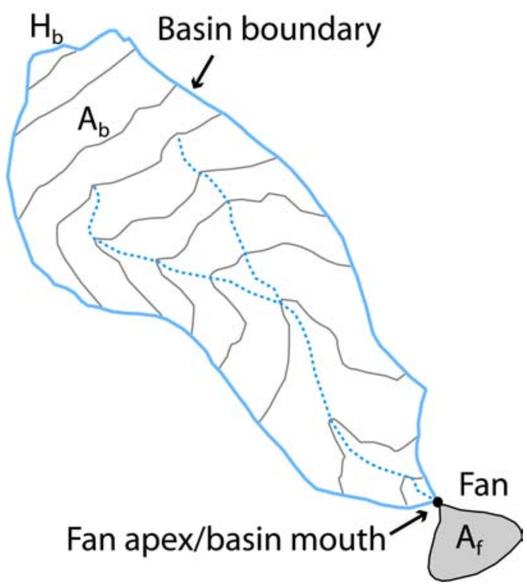

Figure 5. Sketch showing drainage basin and fan morphometry values used in the study. H_b is the drainage basin height and is the difference in elevation between the top of the basin and the fan apex/basin mouth. A_f and A_b are the planimetric area of the fan and basin respectively.

Fan midpoints were chosen and surveyed about one quarter to one third of the way down the fan surface along an approximate bisector of the fan (Melton, 1965, Ryder, 1971). These points were chosen over surveying the distal margin of the fans because many of the fan surfaces have been altered by maintenance activities along I-70. Additionally, the transition between the fan margin and the alluvial flood plain of the Eagle River is indistinct for many of the intact fans. Thus, fan gradient angles calculated from these measurements will reflect the steeper part of the fan as the gradient of the fan generally decreases towards its distal margin. Surveyed positions were determined relative to a stable base point consisting of a length of rebar and aluminum cap that was installed near the center of the study area. The position of the base point was determined using a Federal Base Network Control Station (Station Designation Harrington, PID – AB2471, located about 3 mi (5 km) from the study area along U.S. route 6). All GPS positions were determined using geodetic grade Ashtech* Z-VII and Z-Surveyor GPS receivers and rapid-static and relative positioning surveying techniques (see Van Sickle, 1996 for additional detail).

The surveyed positions were input into a Geographic Information System (GIS) and converted to the NAD27 datum, which allowed the plotting of the points onto a topographic map base. The plotted apex locations provided a reference for delineation of the drainage basin boundaries, which were digitized in the GIS (Figure 1). Maximum elevation for several of the easily accessed smaller basins (F4-F6) was surveyed using GPS. The maximum elevation for the other basins was determined from the USGS 30 m DEM.

Field measurements and mapping were combined with the GPS survey information to estimate the area of several fans that did not appear to have been altered significantly by highway construction or maintenance activities or were large enough that reasonable estimates of area could be made despite the presence of the highway.

Drainage basin morphology was characterized using Melton's number (N_m) defined as

$$N_m = H_b / \sqrt{A_b} \quad , \qquad (1)$$

where H_b is the drainage basin height and A_b is the drainage basin area (Figure 5; Melton, 1965). Melton's number is a dimensionless measure of drainage basin ruggedness. Small, narrow basins with high relief have larger Melton's numbers than broad basins with relatively lower relief. A relation between Melton's number and fan gradient has been used as a tool to classify fans based on dominant process type with some success (Kostaschuk et al., 1986, Jackson et al., 1987, Marchi et al., 1993, Marchi & Tecca, 1995, Coe et al., 1998, Sorriso-Valvo et al., 1998, Parise & Calcaterra, 2000). Surface morphology and sedimentary evidence are used to classify the fans as dominated by either debris flows or fluvial transport. An intermediate or mixed class is frequently identified where the observational evidence points to both processes or is ambiguous (Parise & Calcaterra, 2000, Coe et al., 2003). Melton's number for the drainage basins above the fan is plotted versus the fan gradient and typically the fluvial dominated fans and basins plot in the lower left with small fan gradients and ruggedness values and basins and fans dominated by debris flows plot in the upper right with larger fan gradients and ruggedness values. Glacial history, variability in lithology, tectonic setting, and climate have been identified as factors that may affect the relation between drainage-basin geometry and fan gradient and the effectiveness of using this relation to identify a dominant process type in any given geographic region (Ryder, 1971, Jackson et al., 1987, Parise & Calcaterra, 2000).

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*Any use of trade, firm, or product names is for descriptive purposes only and does not imply endorsement by the U.S. Government.

RESULTS AND DISCUSSION

Table 1 summarizes the morphometric data for the basins and fans measured in the study area. The number-letter combination identifies the fan and drainage basin shown in Figure 1. Drainage basin areas (A_b) were computed in the GIS and range over about two orders of magnitude. The fan gradients (θ_f) range from about 6.5 degrees to almost 19 degrees and generally increase as drainage basin area decreases. Drainage basin heights (H_b) range from 295 ft (90 m) to about 1640 ft (500m). Typically, basins dominated by debris flows have basin ruggedness values (N_m) between 0.3 and 1.5 (Jackson et al., 1987, Bovis & Jakob, 1999), and the studied basins fall within this range indicating a dominance of sediment transport by debris flows.

Table 1. Fan and drainage basin morhpometric parameters. The fan number refers to the locations shown in Figure 1.

Fan number	*Fan gradient (θ_f)*	*Basin height (H_b)*	*Basin area (A_b)*	*Basin ruggedness (N_m)*	*Fan Area*
F1	**6.9 degrees**	**415.76 m**	**131,889 m^2**	**1.14**	**29,200 m^2**
F2	**6.3**	**470.76**	**673,509**	**0.57**	**53,000**
F3	**7.0**	**492.04**	**409,344**	**0.77**	**16,700**
F4	**13.5**	**85.80**	**4590**	**1.27**	
F5	**14.9**	**90.84**	**5329**	**1.24**	
F6	**14.2**	**91.67**	**5481**	**1.24**	**5250**
F7	**11.7**	**98.40**	**4435**	**1.48**	**2920**
F8	**7.4**	**97.98**	**4278**	**1.50**	**2090**
F9	**10.2**	**88.34**	**4464**	**1.32**	
F10	**14.7**	**90.26**	**7273**	**1.06**	
F11	**12.6**	**95.00**	**7258**	**1.12**	
F12	**7.4**	**321.0**	**125,136**	**0.91**	
F13	**14.9**	**88.90**	**7480**	**1.03**	
F14	**18.7**	**92.15**	**5309**	**1.26**	
F15	**15.3**	**94.02**	**7832**	**1.06**	

Melton (1965) reasoned that since Equation 1 combines measures of both the distance and gradient traveled by transported material in a basin it should correlate well with fan gradient. Field data reported from a variety of climatic and geologic settings have shown that fan gradient and basin ruggedness can generally be related by an equation of the form,

$$\theta_f = c(N_m)^b, \quad (2)$$

where c is a constant and b is approximately equal to 1 (Church & Mark, 1980). Figure 6 shows the low correlation (R^2 = 0.21) between Melton's number (N_m) and fan gradient for the basins and fans in the Gypsum study area. Basin ruggedness varies over a narrow range for the smaller basins studied (Table 1), however, the gradients of the fans that lie below the smaller basins vary by a factor of three.

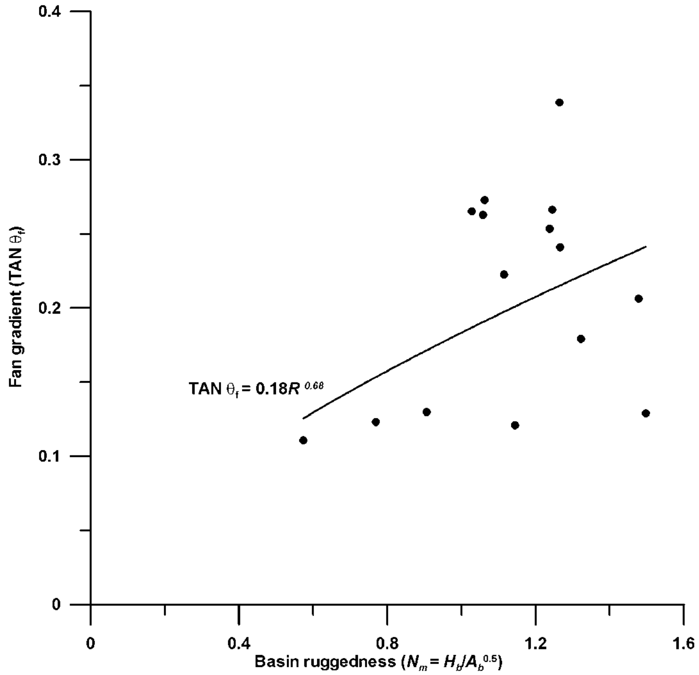

Figure 6. Relation between fan gradient and dimensionless basin ruggedness for the 15 basins and fans in the study area.

The scatter in the relation between θ_f and N_m for the smaller basin-fans may be a function of highway maintenance activities. However, care was taken to make survey measurements in areas that appeared relatively undisturbed, and it is unlikely that the smaller fans, located furthest from I-70 have been affected by human activity. The *relative* error in GPS measurements of elevation between fan apex and midpoint should be less than 0.3 ft (10 cm) yielding a random error of a few tenths of a degree in gradient calculations. The scale of the topographic map (1:24,000) may contribute to error in delineation of the smaller drainage basins resulting in error in the calculation of basin area. However, in the case of basins F4, F5, and F6, the top of the drainage was surveyed and all the smaller basins are completely visible from the ground and easily mapped. We estimate the combined errors in drainage-basin area and basin height result in an error of less than ± 5.0 percent in the calculation of N_m.

The scatter in the morphometric data from the basins and fans in the Gypsum study area may be a function of the erodible Eagle Valley Evaporite that partially underlies the basins at the base of the slopes near the alluvial valley (Widmann, 1997, Lidke et al., 2002). Basins that produce finer grained sediment tend to have alluvial fans with lower slopes (Hooke, 1968).

The Evaporite underlies a proportionally larger area of the smaller basins (Figure 1), and variability in the particle size and rate of colluvium produced from the Evaporite may influence the gradients of the fans. Additionally, several of the smallest basins are essentially large gullies (Figure 2) that drain a flat lying ridge or bench (Figure 1) mantled with a silty regolith. Figure 7 is a photograph of a rill network that has developed at the head of drainage basin/fan F5. The approximate drainage divide is just behind the person in the photograph. The material delivered from these drainage heads to the debris fans may account for some of the scatter in the fan

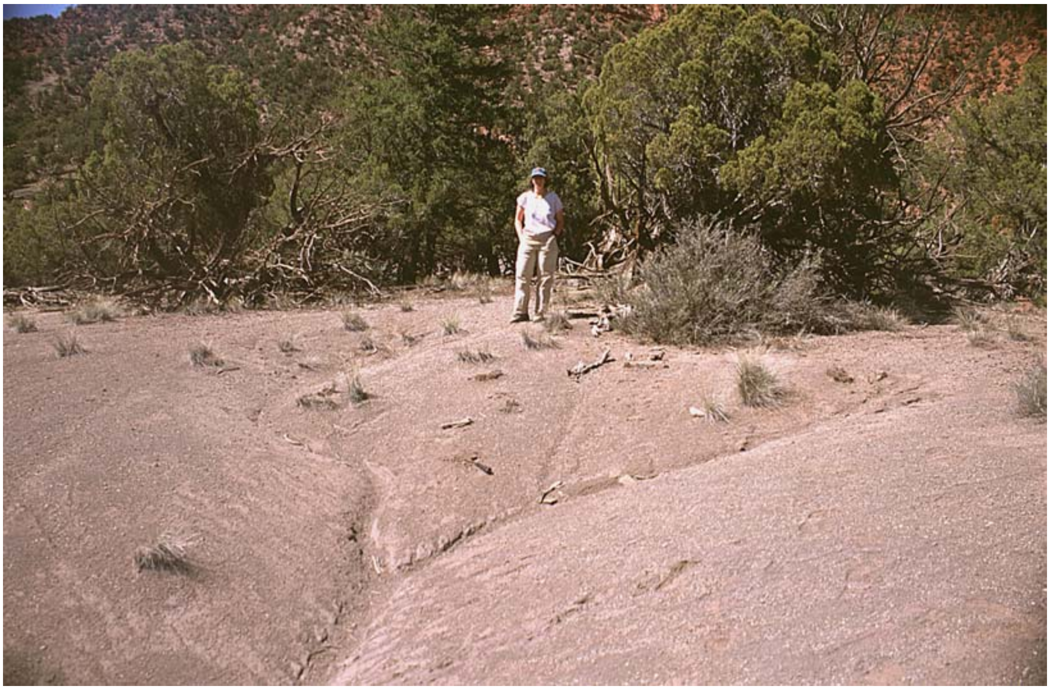
Figure 7. Rill network near the top of drainage basin F5 (Figure 1). Rill networks have developed on the moderate slopes near the drainage divides of several of the smaller basins.

gradient data for the smaller basins. Ryder (1971) reported similar weak relations between basin ruggedness and fan gradient for basins and fans in the Kamloops region of British Columbia and suggested that differences in lithology among the basins in the sample affected the hypothesized power law relation.

The wide range of fan gradients below the smaller basins may also be attributed to processes that transport sediment other than debris flow. Dry ravel, rockfall and other mass movement processes may dominant the deposition on the steeper fans below the smaller basins. Reworking of the fans by overland flow during intense rainfall may also account for some of the variability in fan gradient (Kostaschuk et al., 1986).

Fan area has been estimated for six fans in the study area (Table 1). The areas of the fans are approximate estimates for the largest fans due to the modification of the fan surfaces by highway construction and maintenance activities. Given the limited data set and assuming that fan area exerts first-order control on fan volume, the linear relation between basin ruggedness and fan area shown in Figure 8 ($R^2 = 0.73$) indicates a stronger influence on fan geometry by basin morphology than the data on fan gradient suggest.

The volume of fans dominated by debris flows is primarily a function of the frequency and magnitude of debris flows that deposit material on the fan. Knowledge of the frequency, magnitude, and spatial extent of debris-flow activity is required to assess the hazard on fans. Coe et al. (in press) have shown that debris-flow frequency can be related to basin ruggedness using Melton's number in the Front Range of Colorado. Data from the Gypsum area indicate that basin ruggedness may also be inversely related to fan area and volume. This suggests that

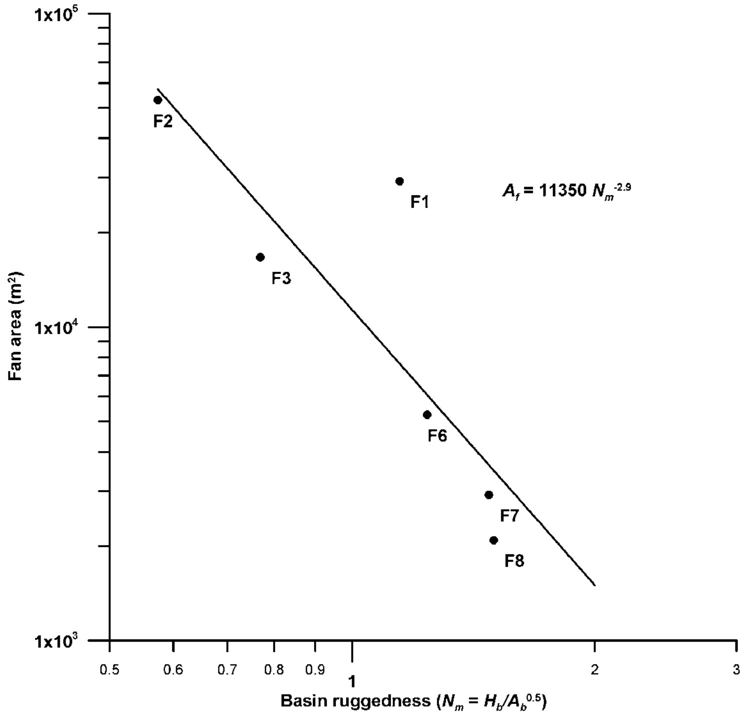

Figure 8. Relation between fan area and dimensionless basin ruggedness for a subset of the basins and fans in the study area.

drainage-basin morphology may be useful for estimating the frequency and magnitude of debris flows from similar small drainage basins in Colorado. Human activity has altered or obscured many of the fans along the I-70 corridor, and detailed investigation of the stratigraphy of intact fans is often impossible. Analysis of drainage basin morphometry is one practical approach that may be applied to assess the debris-flow hazard on these fans. Additional work to estimate debris-flow frequency on the Gypsum fans using historic or stratigraphic information is needed before a hazard assessment can be completed.

CONCLUSIONS

Precise surveying methods and information from a Digital Elevation Model were used to quantitatively analyze the topography and fan geometry for 15 small debris-flow drainage basins and fans near Gypsum, Colorado. Fan gradient showed little correlation with Melton’s number, a dimensionless measure of drainage basin ruggedness. The complex lithology related to late Cenozoic evaporite tectonism, small drainage basin size, debris-flow, and fluvial activity have the effect of introducing significant variation in the gradient of the debris fans. Fan area is inversely correlated with Melton’s number for a subset of the 15 fans suggesting that debris-flow frequency and/or magnitude may be related to drainage basin morphology and that quantitative analysis of drainage basin morphology may be useful in assessing the hazard on fans along the mountainous part of Interstate 70 in Colorado.

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DEBRIS-FLOW RESPONSE OF BASINS BURNED BY THE 2002 COAL SEAM AND MISSIONARY RIDGE FIRES, COLORADO

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Key Terms: debris flow, wildfire, rainfall thresholds, hazard assessments

ABSTRACT

Debris flows can be one of the most hazardous consequences of rainfall on recently burned hillslopes. Understanding the conditions under which debris flows can occur, and characterization of the magnitude of the debris-flow response are critical elements in post-fire hazard assessments. In this study, we use field measurements and observations to document the debris-flow response of basins burned by the 2002 Coal Seam and Missionary Ridge Fires. The Coal Seam Fire burned 12,229 acres in the steep terrain immediately west of Glenwood Springs and the Missionary Ridge Fire burned 72,962 acres just north of Durango. Eyewitness and newspaper accounts of the rainfall-induced runoff events, measurements of channel cross sections, maps of burn severity, and networks of tipping-bucket rain gages are used to develop estimates of the peak discharges of the debris flows and to define the conditions that resulted in the debris flows.

Debris flows were produced from basins underlain by interbedded sandstones, siltstones and conglomerates, and from basins underlain by gneissic quartz monzonite and quartzite. Debris-flow producing basins ranged in size from 0.01 to 8.24 mi^2, had average gradients between 26 and 94 percent and relief ratios between 16 and 73 percent. Basins burned at moderate and high severities over more than 50 percent of their areas were susceptible to debris-flow activity. Nearly 70 percent of the debris-flow generating storms were of durations equal to or less than two hours, and 93 percent of the storms had recurrence intervals of less than or equal to 2 years. The average intensities of the debris-flow triggering storms ranged between 0.04 and 0.65 in/hr, with 10-minute peak intensities up to 2.46 in/hr. Estimates of debris-flow peak discharges between 315 and 5581 ft^3/s were obtained using indirect methods, and values of peak discharge per unit area ranged between 1.0×10^{-5} and 1.2×10^{-3} ft/s. Debris flows with the highest values of peak discharge per unit area occurred in response to storms with average intensities greater than about 0.4 in/hr and with 10-minute peak intensities greater than about 2.0 in/hr. And last, a rainfall intensity-duration threshold for post-wildfire debris flow activity of the form

$I = 0.25D^{-0.5}$, where I = rainfall intensity (in in/hr) and D = the duration of that intensity (in hrs) is defined.

INTRODUCTION AND APPROACH

The fire season of 2002 was extremely active in Colorado. More than 619,000 acres (247,600 ha) were burned by approximately 4600 fires throughout the state (U.S.D.A. Forest Service 2003, National Interagency Fire Coordination Center 2003). Some of the most extensive fire activity included the 137,800-acre (55,120-ha) Hayman Fire, the 12,229-acre (4892-ha) Coal Seam Fire, and the 72,962-acre (29,185-ha) Missionary Ridge Fire (Figure 1).

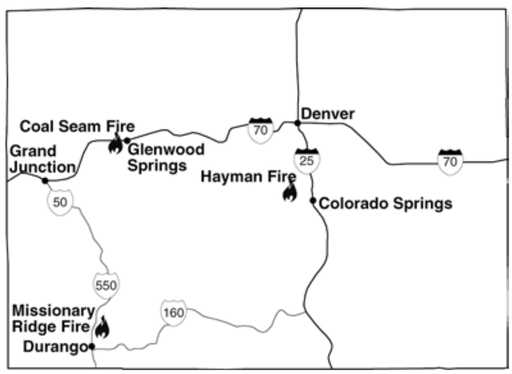


Figure 1. Map showing locations of Hayman, Coal Seam, and Missionary Ridge Fires of 2002 in Colorado

Wildfire can have profound effects on a watershed. Consumption of the rainfall-intercepting canopy and of the soil-mantling litter and duff, intensive drying of the soil, combustion of soil-binding organic matter, and the enhancement or formation of water-repellent soils can result in

Wildfire can have profound effects on a watershed. Consumption of the rainfall-intercepting canopy and of the soil-mantling litter and duff, intensive drying of the soil, combustion of soil-binding organic matter, and the enhancement or formation of water-repellent soils can result in decreased rainfall infiltration into the soil and subsequent significantly increased overland flow and runoff in channels (e.g., Doerr et al. 2000, Martin & Moody 2001, Moody & Martin 2001). Removal of obstructions by wildfire can enhance the erosive power of overland flow, resulting in accelerated erosion of material from hillslopes (Meyer 2002). Increased runoff can also erode significant volumes of material from channels, the net result being the transport and deposition of large volumes of sediment both within and down-channel from the burned area.

Debris flows are frequently produced in response to convective thunderstorm activity over basins burned by wildfire (Parrett 1987, Meyer & Wells 1997, Cannon 2001), as well as in response to winter frontal storms (Morton 1989, Cannon 2000). Debris flows pose a hazard distinct from other sediment-laden flows because of their unique destructive power; debris flows can occur with little warning, can exert great impulsive loads on objects in their paths, and even small debris flows can strip vegetation, block drainage ways, damage structures, and endanger human life. For example, a summer thunderstorm triggered debris flows from the steep basins burned by the 1994 South Canyon Fire on Storm King Mountain, Colorado (Figure 2A) (Kirkham et al.

2000, Cannon et al. 2001). This event inundated nearly 2 miles (3 km) of Interstate 70 with tons of rocks, mud and debris. Thirty vehicles and their occupants were engulfed in the flows, and in two cases, were pushed into the Colorado River. Although some travelers were seriously injured, no deaths resulted from this event. The similarities between the landscapes and geologic materials affected by the Coal Seam and Missionary Ridge Fires and those burned by the South Canyon Fire indicate the potential for a similar runoff response.

The purpose of this report is to document the conditions that resulted in a debris-flow response from basins burned by the Coal Seam and Missionary Ridge Fires. Shortly after each fire was extinguished and before any rainstorms had impacted the area, networks of tipping bucket rain gages were installed, and a series of cross sections was installed in representative basins. After each significant rainfall event, we used field observations to document which basins produced debris flows, sediment-laden floods, and which showed no response. Surveys of channel cross sections made after event-producing storms were used to obtain indirect measurements of peak discharges of the debris flows. By documenting the basin characteristics and burn extent of the debris-flow producing basins, as well as the rainfall conditions that impacted the basins, we are able to define the conditions that lead specifically to the generation of post-wildfire debris flows. Estimates of the peak discharge of debris-flow events are used to define relations between the magnitude of the debris-flow response, storm rainfall triggers and basin area. In addition, a comparison of the rainfall conditions in storms that produced debris flows with those that produced sediment-laden floods or showed no response is used to define the threshold rainfall conditions for the production of fire-related debris flows from similar terrains.

SETTINGS

The Coal Seam Fire burned immediately west of Glenwood Springs, Colorado, and impacted the hillslopes and canyons on both the north and south sides of the Colorado River (Figure 2A). The fire burned through piñon-juniper woodlands, mountain shrublands, and aspen, Douglas fir, and spruce-fir forests. Most of the burned area is characterized by high relief, steep slopes, and tightly confined canyons. Channel gradients range between 20 and 65 percent. The northern-most portion of the burned area is a high elevation, relatively flat plateau known as the Flat Tops. Hillslope gradients range from nearly flat within the Flat Tops to greater than 80 percent within the Mitchell Creek and South Canyon watersheds and on Red Mountain (Figure 2A). The burned area ranges in elevation from approximately 5,720 ft (1787 m) along the Colorado River to 10,400 ft (3250 m) in the Flat Tops.

The area burned by the Coal Seam Fire is underlain primarily by the Pennsylvanian and Permian Maroon formation (consisting of interbedded sandstones, siltstones and conglomerates), a Proterozoic gneissic quartz monzonite, and the upper Cambrian Sawatch quartzite (Kirkham et al. 1997). Smaller extents of dolomite, dolomitic sandstone, shale and limestone have also been mapped in the upper reaches of the basins. A fault that trends east to west just north of the Glenwood Springs Fish Hatchery (Figure 2A) separates the Maroon formation from the quartz monzonite and quartzite. Soils developed on these units are generally shallow, poorly developed and with a high percentage of rock (Cannon et al. 1998). Five samples of the materials that mantle hillslopes underlain by the Maroon Formation are classified as silty sands (SM), and two samples from the quartz monzonites are classified as well-graded sands (SW). Immediately after the fire, the hillslope-mantling soils were observed to be very dry, and even a gentle wind entrained ash and fine sand. Accumulations of loose, unconsolidated dry-ravel deposits up to 1-m thick were observed in many of the tributary drainages to Mitchell Creek, South Canyon, and Basin A; these basins are underlain by the Maroon Formation (Figure 2A). Dry ravel is process frequently observed both during and after fires wherein soils dried during the passage of the fire experience particle-by-particle transport of material downslope by gravity. Dry ravel has been described as an important post-fire process in southern California where channels are loaded with sediment, increasing available sediment for transport in large runoff events (e.g., Wells, 1981). In addition, extensive talus deposits mantle the hillslopes underlain by the metamorphic rock types in tributaries to Mitchell Creek.

The steep channels that drain Red Mountain and the tributary canyons within the Mitchell Creek watershed show evidence of Holocene-to-recent debris-flow activity (Kirkham et al. 1997). These debris flows are most commonly produced from the Maroon formation. An extensive alluvial fan has formed at the base of Red Mountain, and smaller fans are common at the tributary junctions in Mitchell Canyon.

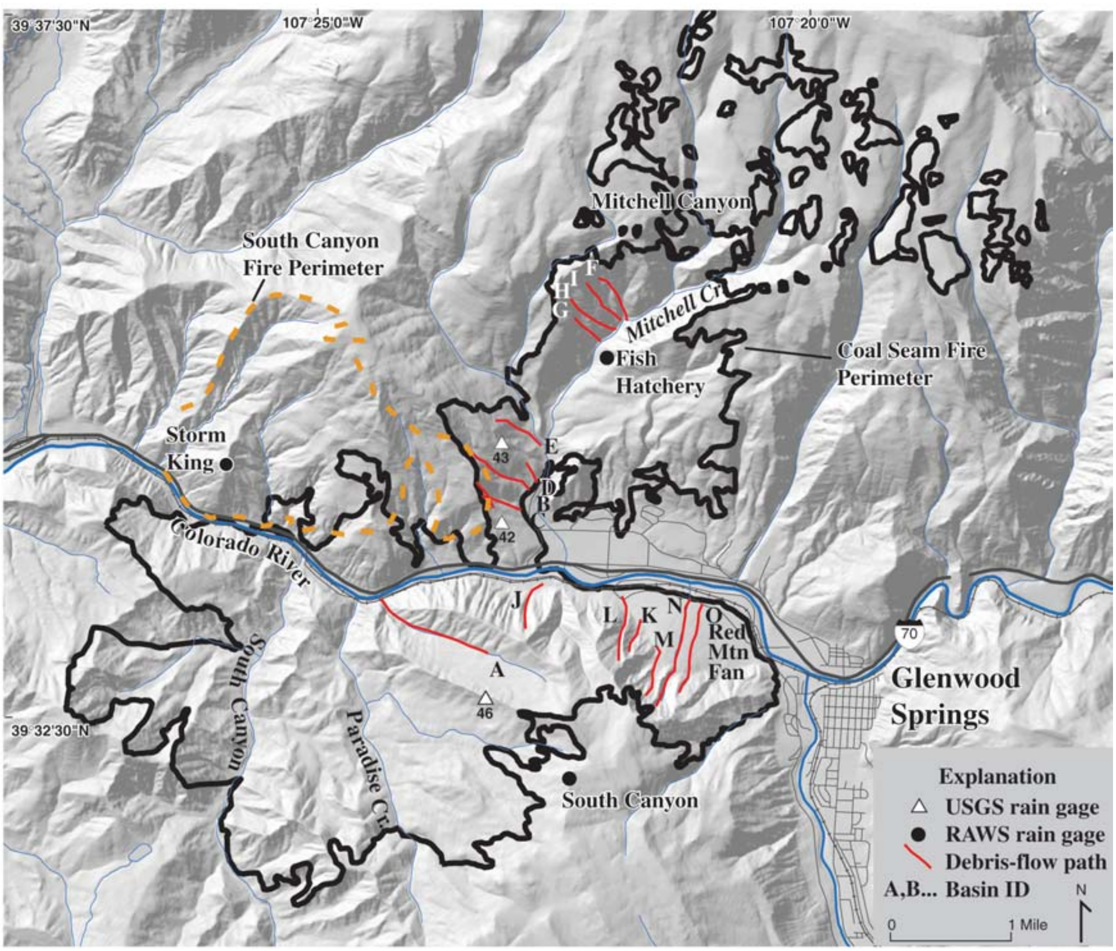


Figure 2A. Shaded relief image showing perimeter of Coal Seam and South Canyon Fires, locations of rain gages, and paths of debris flows generated in response to the August 5, 2002 storm.

Glenwood Springs has a semi-arid climate with low humidity throughout the year (Interagency Burned Area Emergency Response Team 2002). Average high temperatures in the valley bottoms range from 30 to 40 degrees F. in winter to 80 to 90 degrees F. in the summer months.

Average annual precipitation in the valley is between 15 and 17 in (381 to 432 mm), and up to 38 in (965 mm) at higher elevations. Precipitation usually falls during two periods – either as winter frontal storms, or summer convective thunderstorms. The thunderstorms are characterized by localized, short duration rainfall.

The Missionary Ridge Fire burned north and northeast of the city of Durango, Colorado, and included portions of the Animas, Florida and Los Pinos River Valleys (Figure 2B). The fire burned Ponderosa pine, mixed conifer, aspen and spruce-fir forests at elevations ranging from approximately 6500 ft (2030 m) along the Animas Valley to approximately 11,400 ft (3560 m) in the northern portion of the fire. Most of the burned area is characterized by steep hillslopes and canyons with gradients between 14 and 30 percent.

The lower Permian Cutler Formation underlies most of the area burned by the Missionary Ridge Fire, and is comprised of interbedded sandstones, siltstones and conglomerates (Carroll, et al. 1997, Carroll, et al. 1998, Carroll, et al. 1999, Gonzales, et al. 2002). The Hermosa, Molas, Junction Creek, Wanakah, Entrada, Dolores and Morrison Formations, which consist of interbedded sandstones, shales, limestones and conglomerates, and the underlying, older Eolus Granite have also been mapped within the burned area. In addition, extensive deposits of Quaternary glacial and colluvial deposits are mapped in many of the tributary drainages to the Animas, Florida, and Los Pinos Rivers. Soils within the burned area are most commonly alfisols – a forest soil with an illuviated clay horizon (Burned Area Emergency Response Team 2002).

Although dry-ravel deposits were observed in some basins, they were not as extensive or as thick as those observed in the basins burned by the Coal Seam Fire.

Landslide and debris-flow activity is common in the area (Burned Area Emergency Response Team 2002). Debris flows are frequently generated from the Cutler Formation and Eolus Granite, and with less frequency from the Hermosa and Morrison Formations. Active, large alluvial fans have developed at the mouths of tributaries to the Animas River along the mountain front and to a lesser extent on other areas.

The area around Durango has a semi-arid climate with generally warm summers and cold winters. Annual precipitation in Durango is 18.6 in (472 mm), and winter snowfall totals average about 70 in (1778 mm) (Burned Area Emergency Response Team 2002). Forty-two percent of the precipitation in Durango falls between August and October during the summer-fall monsoon season. The monsoon season is characterized by severe, but locally variable and short-lived, thunderstorms.

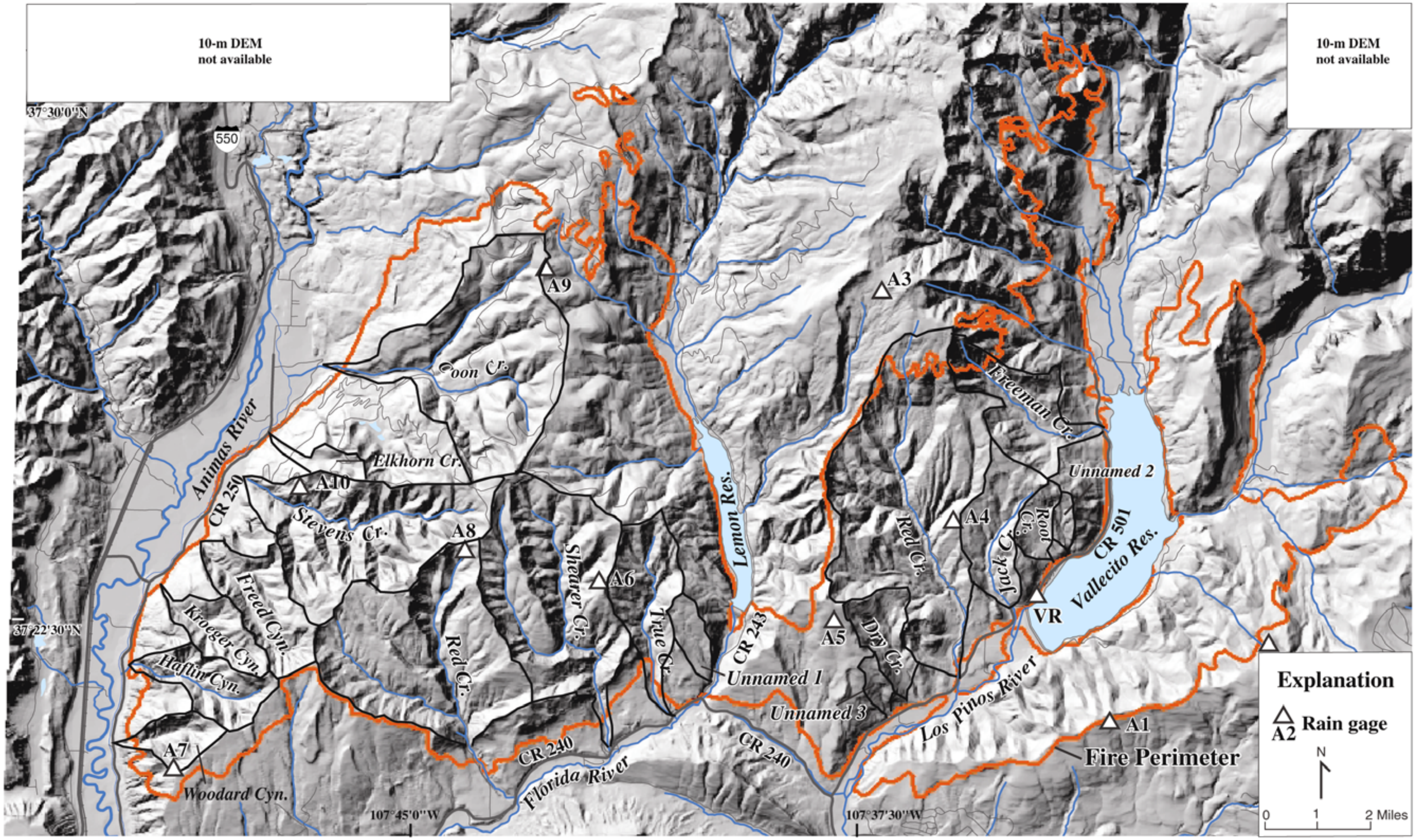


Figure 2B. Shaded relief image showing perimeter of Missionary Ridge Fire and locations of rain gages.

METHODS

To document the debris-flow response of the Missionary Ridge and Coal Seam Fires, we utilized a network of tipping-bucket rain gages installed within the burned areas shortly after each fire was extinguished and before any rainstorms had impacted the areas. Twelve tipping-bucket rain gages were installed by the U.S. Geological Survey Water Resources Division in the area burned by the Missionary Ridge Fire as part of the post-fire Burned Area Emergency Rehabilitation monitoring program (Burned Area Emergency Response Team 2002) (Figure 2A). Data from these gages are available on the web at: http://co.water.usgs.gov/fires/missionridge. We installed three tipping-bucket rain gages in the area burned by the Coal Seam Fire (Figure 2B). The rain gages recorded the date and time of each 0.01-inch accumulation of rainfall, and from these data, we extracted measures of storm duration, total storm rainfall, average storm intensity, and peak 10-, 15-, 30- and 60-minute intensities. In addition, four Remote Access Weather Stations (RAWS) installed and maintained by the U.S.D.A. Forest Service and Bureau of Land Management within the Coal Seam Fire were used to supplement the information recorded by the tipping bucket rain gages (Figure 2A). Although these weather stations recorded rainfall totals (in addition to other weather data) at 10-minute intervals, some data were available only at 1-hour intervals.

After each significant rainfall event, we used field observations, police dispatch records, newspaper accounts, and observations from eye witnesses to document which basins produced debris flows, sediment-laden floods, and which showed no response. We defined threshold rainfall conditions that can result in fire-related debris-flow activity by comparing measures of rainfall intensities and durations recorded by gages located within 1 mi (1.6 km) of debris-flow producing basins with the those measures recorded by gages within 1 mi (1.6 km) of basins that produced only sediment-laden floods or showed no response. The threshold is defined by visually distinguishing those rainfall intensities and durations that occurred during the debris-flow producing storms.

Surveys of channel cross sections installed in basins in both fires either prior to any significant rainfall accumulations, or immediately after event-producing storms, were used to obtain indirect measurements of peak discharges of the debris flows. The cross-sectional areas defined by the passage of each of the debris flows were measured at either two or three locations near the mouths of eight basins in the Coal Seam Fire and ten basins in the Missionary Ridge Fire. Estimates of peak discharges of the debris-flow events were obtained using a slight variation of the velocity-area method described by O'Connor et al. (2001) wherein the product of the average of the two or three channel cross-sectional areas at peak stage and an estimate of flow velocity is calculated. Maximum flow stage was determined by the highest evidence of inundation at the cross section, with either a prominent muddy veneer or levees. The cross-sectional areas were measured by installing two pieces of steel rebar perpendicular to the channel with markings to indicate a level line, stretching a tape measure across the channel from the right to left banks, anchoring it at the level lines on the rebar, and recording the depth to the channel bed at 0.5-m intervals. In an effort to obtain velocities at a conveyance reach, the cross sections were located on fairly straight reaches of channel with constant gradients that showed little evidence of either erosion or deposition, other than the muddy veneer or the levees. For the debris flows that originated from basins underlain by sedimentary rock types, the average debris-flow velocity was assumed to be 16 ft/s (5 m/s). Debris flows that originated from the gneissic quartz monzonite and quartzite in the Coal Seam Fire appeared to be more viscous, and thus slower-moving, and we assumed an average velocity of 10 ft/s (3 m/s) for these flows. The assumed velocity of 16 ft/s (5 m/s) is based on an average of eight velocities calculated using the super-elevation method for debris flows generated from basins burned by the 1994 South Canyon Fire (Cannon et al. 1998). This average velocity is well within the range of 10 to 19 ft/s (3 to 6 m/s) found for debris flows in alpine environments by O'Connor et al. (2001). Although the critical-flow indirect method might provide a better measure of peak discharge of debris flows than the velocity-area method (O'Conner et al. 2001), no sites suitable for this approach could be located, and the values presented here should be viewed at best as estimates.

Basins within the burned areas were delineated using 10-m Digital Elevation Models (DEMs) and the watershed delineation tools in Arc 8.2. Measures of basin area, average basin gradient, and the relief ratio for each basin were then obtained from the DEM. Average basin gradient was calculated as the average of the gradient of each of the grid cells within the basin. The relief ratio was measured as the elevation change from the basin mouth to the drainage divide divided by the length of the longest channel extended to the drainage divide.

The area of each basin burned at varying severities was extracted from maps of burn severity generated using the Normalized Burn Ratio (NBR), a method that utilizes Landsat Thematic Mapper data (Key & Benson 2000). Burn severity is a relative measure of the effects of fire on soil hydrologic function (U.S. Department of the Interior, 2001). Areas classified as high burn severity generally exhibit complete consumption of the forest litter and duff, evidence of heating of the soil surface, and combustion of all fine fuels in the canopy. Areas burned at moderate severity can be characterized by the consumption of litter and duff in discontinuous patches, and leaves or needles, although scorched, may remain on trees. Areas of low burn severity may show charring of the relatively intact litter and duff, intact fine roots within the soils, and very little effect of fire on the canopy.

In this paper, we first describe the response of the burned basins in each of the fires to the summer of 2002 rainfall. We then present the conditions in the debris-flow producing basins, including lithology, basin area and gradient, and burn severity, followed by our estimates of debris-flow peak discharge. We next describe the rainfall conditions that resulted in debris-flow activity, and examine relations between these peak discharges, basin area, and debris-flow producing rainfall intensities. And last, we define the threshold rainfall conditions that resulted in the generation of debris flows from the burned basins.

SUMMER OF 2002 RESPONSE

In the following section, we have compiled a summary of the erosional response of individual burned basins in both fires to the 2002 summer monsoon season. We describe which basins showed a debris-flow response, and which produced sediment-laden floods. Those basins not mentioned in the summary either showed no response, or we have no record. Where available, we include information about the character of the debris-flow deposits, the channels through which the debris flows passed, and the timing of the events. Storm rainfall characteristics recorded from rain gages located within 1 mi (1.6 km) of the responding basin are shown in Tables 1A and B, and the estimates of debris-flow peak discharges that we were able to obtain are shown in Tables 2A and B. This compilation depended on field notes, notes of eyewitness interviews, and reviews of newspaper accounts. We apologize if we have missed or misrepresented events.

Coal Seam Fire

August 5, 2002 - Debris flows were first produced from basins burned by the Coal Seam Fire in response to the first heavy rain fall of the monsoon season. Field reconnaissance immediately following the event indicated that at least 15 basins produced debris flows in response to this storm (Figure 2A). The road along Mitchell Creek was blocked with an estimated several hundred cubic meters of debris-flow material in numerous places, and a 6-ft (2-m) diameter culvert was completely blocked by debris produced from Basin A. The debris flows spread over the fan surfaces, leaving deposits approximately 1 ft (0.3 m) thick that consisted of isolated clasts up to 1.5 ft (0.5 m) in diameter in abundant mud and ash (Figures 3A and B). The channels of Basins B, C, and D were flushed of the pre-existing dry-ravel deposits and eroded as much as about 2 ft (0.6 m) in depth, exposing bedrock in many places along their lengths. Debris flows were also produced from basins above the Fish Hatchery (Figure 4), from basins that supply the Red Mountain fan (Figure 5), and from Basin A. The debris flows produced from the basins above the Fish Hatchery deposited levees up to 2.0 ft (0.6 m) high lining the channels and lobes of material up to 5 ft (1.6 m) high at the flow terminus. The debris-flow materials consisted primarily of gravel and cobbles in a muddy matrix.

This storm dropped between 0.16 and 0.67 inches (4 and 17 mm) of rain on the burned area between 8:50 and 10:00 p.m., and most of this rain fell in just 10 minutes (Table 1A). The sheriff's dispatch records reported that at 9:05 p.m. a car was trapped by a debris flow flowing out of Basin B, just 5 minutes after the onset of the storm, as recorded by Rain Gage 43 (Figure 2A). In contrast, a train was reported stuck in mud flowing out of Basin A at 9:42 p.m., nearly an hour after the storm started.

Sept. 7, 2002 – Basins F, G, H, and I in the upper Mitchell Creek watershed produced debris flows consisting of up to gravel-sized material in a muddy matrix. The deposits left by these events appeared to have had higher water contents, relative to sediment loads, than the earlier flows.

Sept. 11, 2002 - Basins F, G, H, and I in the upper Mitchell Creek watershed again produced debris flows. A witness reported hearing two roars during the storm, indicating that there were either two pulses of debris flow in one basin or multiple basins producing debris flows at different times. During this storm, Mitchell Creek was observed to be heavily sediment-laden and boulders up to 15 in (40 cm) in diameter were transported downstream. No flooding or debris flows were reported or observed in any other of the burned basins, although they received somewhat more rainfall than the debris-flow producing basins.


A.


B.

Figures 3A and B. Photographs of A) debris-flow paths and B) deposits produced during August 5, 2002 storm from Basins B and C, Coal Seam Fire.

Figure 4. Debris-flow deposit produced during August 5, 2002, storm from Basin H, Coal Seam Fire.

Figure 5. Debris-flow paths (lighter colored material) from the August 5, 2002, storm on the Red Mountain Fan. Buildings in upper right for scale.

Sept. 12, 2002 - A basin located on the Eastern flank of Red Mountain flooded and made newspaper headlines in Denver, Colorado. This flood flowed through a residential area and many basements were flooded with water and mud.

Sept. 17, 2002 – Debris flows were again produced from basins F, G, H, and I above the Fish Hatchery in Mitchell Creek.

Oct. 2-3, 2002 – Debris flows were again observed emanating from basins F, G, H, and I above the Fish Hatchery, and Basins B and C showed evidence of the passage of sediment-laden floods. Up to 3.3 ft (1.0 m) of incision was measured at the mouth of Basin C, and up to 2.3 ft (0.7 m) at the mouth of Basin B. These events occurred in response to a storm that lasted more than 10 hrs with an average intensity of 0.09 in/hr (2 mm/hr) (Table 1A). Although several short bursts of heavy rainfall within this storm with peak intensities between 0.11 and 0.60 in/hr (3 to 15 mm/hr) were recorded, the time of occurrence of the debris flows within the storm is not known with sufficient accuracy to link their occurrence to high intensity rainfall.

Missionary Ridge Fire

July 11, 2002 – The first storm to impact the area burned by the Missionary Ridge Fire was a localized thunderstorm that impacted the area south of the Vallecito Reservoir dam. The Durango Herald (2002) reported that La Plata County Road (LPCR) 501 and a softball field were flooded with ash and mud flushed from the hillslopes. This storm occurred before the rain-gage network had been installed; thus no record is available.

July 22, 2000 – This storm similarly resulted in a flush of ash and mud from the burned hillslopes near Vallecito Reservoir. LPCR 501 was closed between the Vallecito Reservoir dam and the intersection of LPCR 240 (Figure 2B) in order to clear ash, mud, debris, and a 400 lb (180 kg) boulder that was moved by the flood waters (Durango Herald, 7/23/2002). Red Creek also showed significant flood activity.

July 23, 2002 – The first significant storm of the season impacted the Florida River basin. Debris flows were produced from an unnamed basin just below Lemon Reservoir (Unnamed Basin 1, Figure 2B), and significant sediment-laden floods were produced from Dry, Shearer and True Creeks. At least five vehicles, four containing people, were swept off LPCR 240 by the advancing water (Durango Herald, 2002). The debris flow from Unnamed Basin 1 crossed LPCR 243, blocked a driveway, and damaged a guardrail on a bridge. Material consisting of ash, fine-grained sand, gravel, and clasts up to 3 ft (1 m) in diameter was deposited up to 4 ft (1.2 m) deep on the road. This storm also resulted in sediment-laden floods that crossed LPCR 501 along Vallecito Reservoir in several places.

August 3, 2002 - This storm resulted in the generation of debris flows from many basins within the Los Pinos River drainage and along LPCR 240 (Figure 2B). A debris flow was produced from Freeman Creek at about 12:15 pm. Material up to 6 ft (2m) in diameter was moved in this event, and passage of the flow resulted in nearly 5 ft (1.5 m) of incision of the channel bed. Debris flows crossed LPCR 501 in several places between Freeman Creek and Unnamed Basin 2 (Figure 2B). Root Creek also produced a debris flow that deposited material approximately 10 ft

(3 m) deep including boulders up to 6 ft (2 m) in diameter at the junction with LPCR 501. Debris flows also crossed LPCR 240 in several places.

August 5, 2002 – Debris flows were again reported to have been produced from Unnamed Basin 1. Materials up to 4 ft (1.2 m) in diameter in abundant muddy matrix crossed CR 243. Sediment-laden streamflow events occurred along LPCR 501 near Vallecito Reservoir.

August 8, 2002 – This storm brought more rain to the Florida and Los Pinos River basins. Unnamed Basin 2 produced a debris flow that deposited ash, mud, and debris in a business located across LPCR 501. This event moved material up to 3 ft (1 m) in diameter across the road. Sediment-laden streamflow was produced from many of the smaller drainages north of Unnamed Basin 2, and flooding in Freeman Creek led to additional incision in the channel bed. Unnamed Basin 3 produced a debris flow that deposited levees approximately 3 ft (1 m) high that lined the channel; material up to 3 ft (1 m) in diameter was transported in a muddy matrix by this event. Adjacent Dry Creek showed evidence of sediment-laden streamflow. Root Creek also experienced another debris flow, and a sediment-laden flood was produced by Elkhorn Canyon along the Animas River. Material from the Elkhorn Canyon event covered LPCR 250 with about 1 ft (0.3 m) of ash and mud.

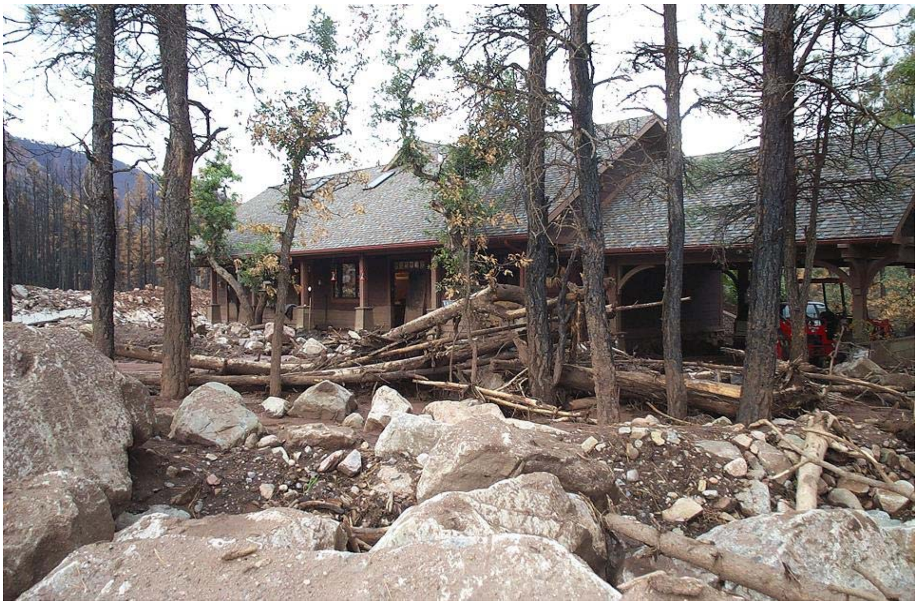
August 20, 2002 – This storm resulted in the generation of sediment-laden floods from Freeman and Dry Creeks, as well as from Unnamed Basin 3.

August 21, 2002 – An apparently localized storm resulted in the production of another debris flow from Root Creek, and light flooding in adjacent drainages. Passage of the flow resulted in approximately 3 ft (1 m) of channel incision, and significant deposition along the channel banks. Material up to 3 ft (1 m) in diameter was transported by this event. Deposits from sediment-laden floods were also observed at the mouth of Freeman Creek.

August 29, 2002 – A localized rainstorm triggered debris flows from Haflin and Kroeger Canyons in the Animas Valley. The flows carried material no greater than 3 ft (1 m) in diameter in a muddy matrix.

September 7, 2002 – This was the first big storm to impact the basins that drain into the Animas River. Debris flows were produced from Coon Creek, Stevens Creek, Freed Canyon, and Woodard Canyon. These events blocked CR 250 in three places with several feet of debris. The event on Stevens Creek covered approximately one-quarter of the fan area with deposits up to 5 ft (1.5 m) deep in places, and inundated a home that prior to the event was at least 200 ft (62 m) from the active channel with tons of rock and mud (Figures 6A and B). This event moved material up to 8 ft (2.5 m) in diameter in a muddy matrix. Evidence on the fan surface suggests that channels were blocked by large boulders and diverted many times during the event. Upstream from the fan, the channels showed up to 8 ft (2.5 m) of incision into extensive valley-fill deposits. The debris flow produced from Freed Canyon moved materials up to 6 ft (2 m) in diameter in a muddy matrix. The high mud line produced from this event was measured 12 ft (4 m) above the post-event channel base, and up to 6 ft (2 m) above the original bank levels. The area downstream from the waterfall at the head of the fan was heavily scoured in this event. Deposition by the debris flow in Coon Creek resulted in the shifting of the channel bed to the north by about 10 ft (3 m). Material from this event consisted of up to cobble-sized material in a muddy matrix. In addition to these events, sediment-laden floods near Red, Shearer, and True Creeks blocked CR 240.


A.


B.

Figures 6A and B. Photographs of A) debris-flow path near apex of Stevens Creek Fan, and B) debris-flow deposits on fan, Missionary Ridge Fire. Debris flows occurred during September 7, 2002, storm.

September 10-12, 2002 – This storm had the longest duration of any that occurred during the 2002 monsoon season. The storm started on the evening of September 10 and continued into the early morning hours of September 12th. Debris flows were again produced from Coon and Stevens Creeks, and Woodard Canyon, but these were smaller discharge events carrying smaller (up to cobble-sized) material than those of September 7. Deposition by debris flow on Coon Creek filled in the channel, and shifted the channel back to the south. Soil slip scars that formed during this storm were observed in the upper reaches of Stevens Creek. These scars were relatively small – at most 100 ft^2 (10 m^2) in area. Material from these scars mobilized into debris flows that traveled a few hundreds of feet down the hillslopes on which they originated. Debris flows were also produced from Root Creek near Vallecito Reservoir and from Unnamed Basin 1 near Lemon Reservoir. Sediment laden floods also resulted in the closure of LPCR 501 at Jack and Dry Creeks near Vallecito Reservoir.

September 20, 2002 – A storm focused on the southern end of the Animas Valley front produced debris flows from Haflin and Kroeger Canyons and from Stevens Creek. These events carried materials only up to about 1.5 ft (0.5 m) in diameter, a significant decrease from previous events. These events appear to have been triggered by a fairly localized storm, indicated by rain gages A7 and A12 (those located nearest to the canyons), which did not record any rainfall on this date.

October 2, 2002 – The last of the monsoon storms to impact the area produced sediment-laden floods on Coon Creek. These events reworked the existing deposits and scoured the south side of the channel.

DEBRIS-FLOW PRODUCING BASINS

In this section we document some of the conditions that produced debris flows in response to the summer 2002 rainfall. The parameters we examine include lithology, basin area, average basin gradient and relief ratio, and burn extent. Other conditions may certainly affect debris-flow occurrence from recently burned basins; here we examine these parameters list as possible first-order effects.

Lithology

Most of the debris flows observed in the two burned areas were produced from basins underlain by the interbedded sandstones, siltstones, and conglomerates of the Maroon and Cutler Formations (Tables 3A and B). These formations are also the most extensive in the two burned areas; thus, this result was not unexpected. However, debris flows were also produced from basins underlain by quartz monzonites and quartzites in the Coal Seam Fire, and from other sedimentary units in the Missionary Ridge Fires. Samples of the materials that mantle hillslopes underlain by the Maroon Formation are classified as silty sands (SM), and materials from the quartz monzonites are classified as well-graded sands (SW).

The availability of readily entrained materials mantling hillslopes and infilling channels appears to affect how frequently debris flows are produced. Although the Cutler and Maroon Formations are lithologically similar, multiple debris flows were produced throughout the monsoon season from basins underlain by the Cutler Formation (Missionary Ridge Fire), while basins underlain by the Maroon formation (Coal Seam Fire) produced only one large debris-flow event. Later storms with similar or greater intensities to the debris-flow producing storm impacted the area burned by the Coal Seam Fire, so this difference cannot be attributed to meteorological vagaries. Rather, field observations in the basins burned by the Coal Seam Fire prior to the onset of summer thunderstorms indicated that although an abundance of dry-ravel deposits lined the channels, bedrock was usually less than about 2 ft (0.6 m) below the channel surface. Repeat surveys of a series of cross sections installed along the length of three channels indicated that most of the material incorporated into the debris flows was the channel-lining deposits, but significant channel incision did not occur (Gartner et al. in prep.). In this case, sufficient material to generate repeat debris flows was not available. In contrast, geologic mapping and field observations in the area burned by the Missionary Ridge Fire showed thick colluvial and glacial fills within many of the basins (Carroll et al. 1997, Carroll et al. 1998, Carroll et al. 1999, Gonzales et al. 2002). Field observations of up to 8 ft (2.5 m) of incision into these deposits, and a lack of significant hillslope erosion suggest that most materials in the Missionary Ridge debris flows originated from these deposits and that these deposits provided an ample material source for repeat debris-flow activity. In addition, the extensive talus deposits that mantle the hillslopes underlain by metamorphic materials in the Coal Seam Fire provided an abundant source of materials for repeat debris-flow activity. This finding indicates that debris-flow susceptibility cannot be simply evaluated in terms of underlying bedrock lithology or soils; understanding and characterization of the availability of readily eroded material is also necessary in a comprehensive hazard assessment.

Basin Area and Gradient

Debris flows were produced from basins with broad ranges in area, average gradient, and relief ratio. Those basins that produced debris flows burned by the Coal Seam Fire were between 0.01 and 0.83 mi^2 (0.03 and 2.15 km^2) in area, ranged in average gradient between 46 and 94 percent, and had relief ratios between 24 and 73 percent (Table 3A). Debris-flow producing basins in the Missionary Ridge Fire ranged in area between 0.25 and 8.24 mi^2 (0.64 and 21.34 km^2), in average gradient between 26 and 58 percent, and in relief ratio between 16 and 30 percent (Table 3B).

Basin area/relief ratio threshold – Measures of the areas and relief ratios of the debris-flow producing basins within the Coal Seam and Missionary Ridge Fires, combined with those measured from other basins throughout the western U.S. that also produced debris flows (Cannon 2001), are used to define the basin conditions most likely to produce debris flows (Figure 7). Basins with areas and relief ratios that fall above the threshold line shown in Figure 7 are those most likely to produce post-wildfire debris flows, given sufficient rainfall.

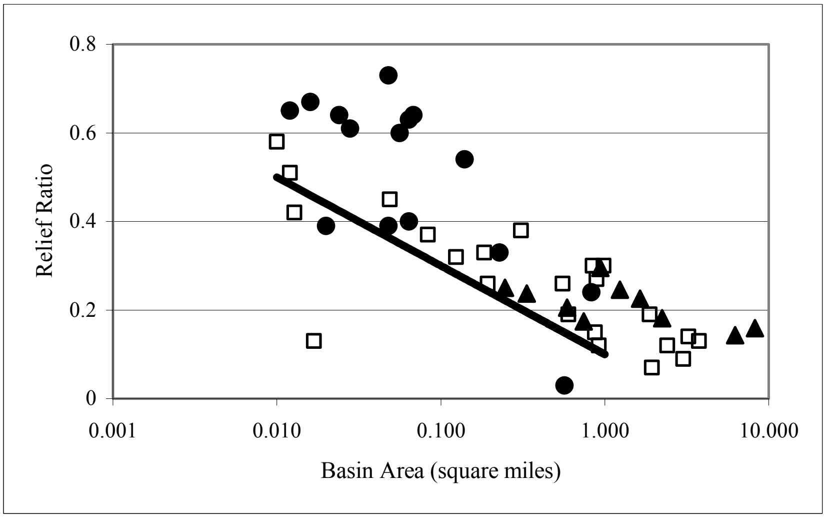


Figure 7. Basin area and relief ratios measured for basins that produced debris flows from the Coal Seam Fire (filled circles), the Missionary Ridge Fire (filled triangles), and fires located throughout the western U.S. (Cannon, 2001) (open squares). Solid black line defines threshold conditions for most basin areas and relief ratios known to have produced debris flows, given sufficient rainfall.

Burn Severity

Debris flows were produced from basins with as little as 3 percent of the area burned at high severity in the Coal Seam Fire, and from basins with as little as 22 percent of the area burned at high severity in the Missionary Ridge Fire (Tables 3A and B). Thus, debris flows can be produced from basins that have experienced very little high-severity fire. However, if we look at the combination of areas burned at high and moderate severities within a basin, we see that between 54 and 100 percent of the areas of the debris-flow producing basins were burned at this combination (Tables 3A and B). This suggests that a threshold value of around 50 percent of the basin area burned at high and moderate severities might be a good indicator of post-fire debris-flow susceptibility, again given sufficient rainfall.

PEAK DISCHARGE ESTIMATES

Peak discharge estimates measured for debris-flow events generated from basins burned by the Coal Seam Fire ranged between 347 and 781 ft^3/s (10 and 22 m^3/s) (Table 2A), and those estimated for the debris flows generated from the basins burned by the Missionary Ridge Fire were considerably higher, ranging between 293 and 5581 ft^3/s (9 and 167 m^3/s) (Table 2B). Normalizing the estimated peak discharges for the Coal Seam Fire by basin area results in values between $2x10^{-5}$ and $1.24x10^{-3}$ ft/s ($6.5x10^{-6}$ and $3.8x10^{-4}$ m/s) (Table 2A); these values are generally higher than those calculated for the Missionary Ridge Fire, which are between $1.0x10^{-5}$ and $1.5x10^{-4}$ ft/s ($2.0x10^{-6}$ and $4.4x10^{-5}$ m/s) (Table 2B).

DEBRIS-FLOW TRIGGERING STORMS

Nearly 70 percent of the storms that generated debris flows from the basins burned by the Coal Seam and Missionary Ridge Fires were of durations equal to or less than two hours, and ranged in intensities between 0.04 and 0.65 in/hr (1.0 and 16.5 mm/hr) (Tables 1A and B, Figure 8). With the exceptions of the October 2-3, 2002 storm recorded by the Rain Gage 43 in the Coal Seam Fire (recurrence interval of 5 yrs), and the July 7, 2002 storm recorded by gage A6 in the Missionary Ridge Fire (recurrence interval of 10 years), (Miller et al. 1976), all of the debris-flow triggering storms had recurrence intervals of less than or equal to two years (Tables 1A and B). Some eyewitnesses reported that the debris flows occurred in response to periods of high-intensity rainfall during the storm. The 10-minute peak intensities recorded near the debris-flow producing basins varied over an order of magnitude, and ranged between 0.24 and 2.46 in/hr (6.3 and 62.5 mm/hr) (Tables 1A and B).

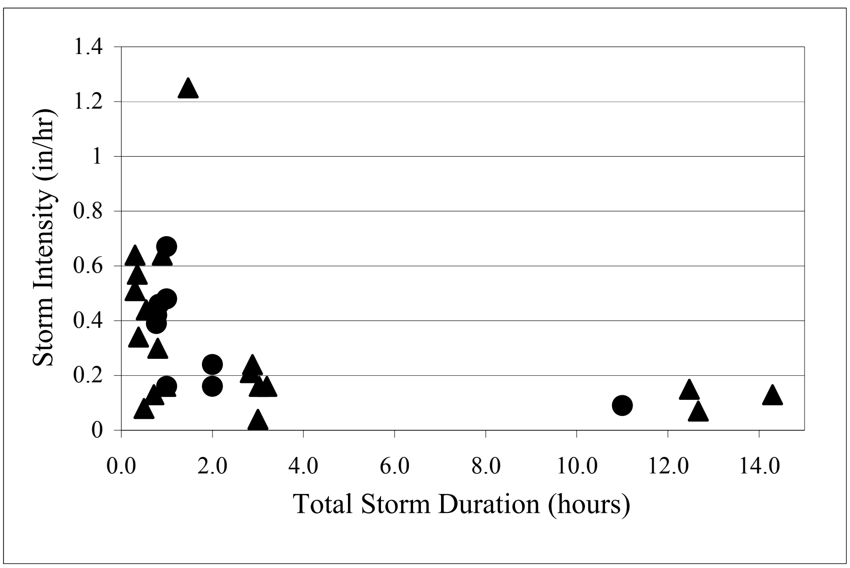
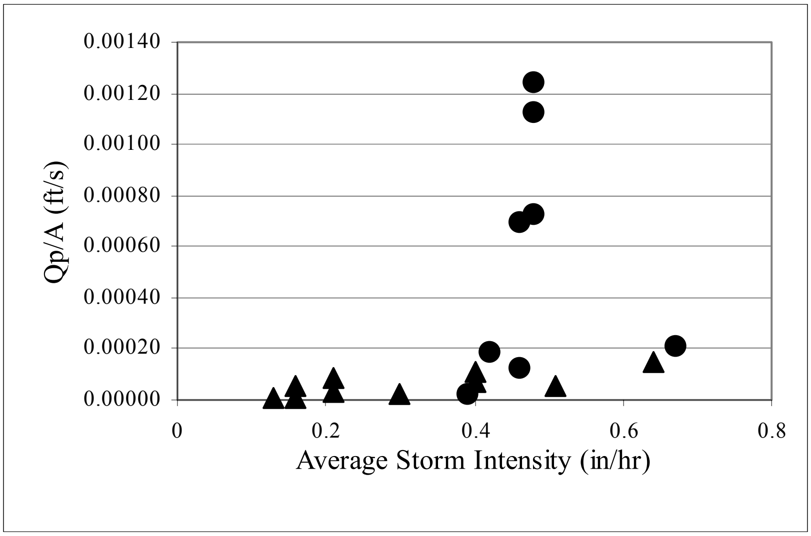


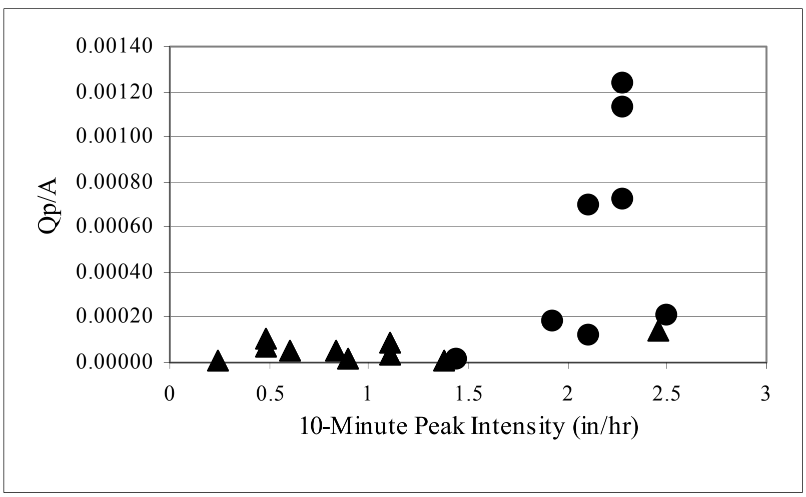
Figure 8. Average storm intensity and duration of debris-flow producing storms. Circles are measurements from storms that triggered debris flows from basins burned by the Coal Seam Fire and triangles are measurements from storms that triggered debris flows from basins burned by the Missionary Ridge Fire.

Peak Discharge, Basin Area and Rainfall Intensity

Relations between the magnitudes of the debris-flow response, basin area, and storm rainfall triggers are shown in Figures 9A and B. The debris flows with the highest values of peak discharge normalized by basin area, and thus the most potentially destructive, occurred in response to storms with average intensities greater than about 0.4 in/hr (10 mm/hr) and 10-minute peak intensities greater than about 2.0 in/hr (50 mm/hr).



A.



B.

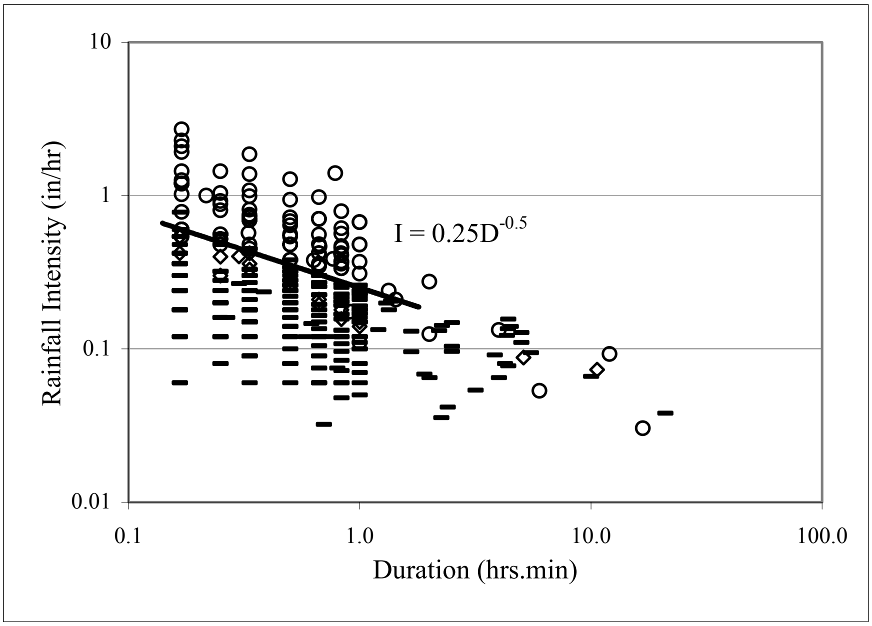
Figures 9A and B. Relations between estimates of debris-flow peak discharge normalized by basin area and (A) average storm intensity, and (B) 10-minute peak intensity for the Coal Seam Fire (circles) and the Missionary Ridge Fire (triangles).

RAINFALL INTENSITY-DURATION THRESHOLD FOR POST-FIRE DEBRIS-FLOW ACTIVITY

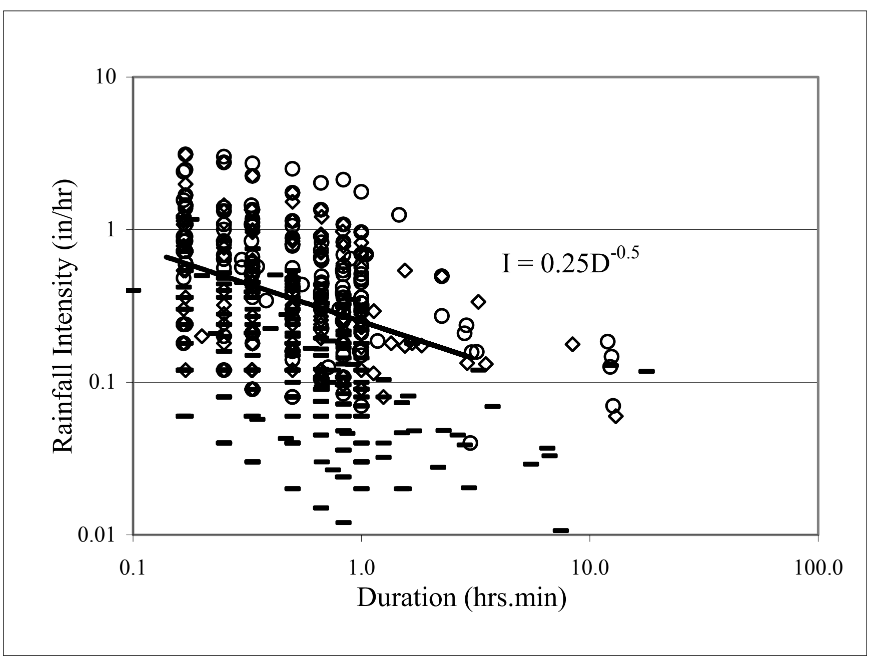
Although a number of workers have described the triggering of fire-related debris flows in response to high intensity rainfall (e.g., Cleveland 1977, Wells 1981, Parrett 1987, Booker 1998, Cannon et al. 1998), little work has been done to define the threshold rainfall conditions that result in debris flows. Such a threshold is a useful tool in issuing warnings and planning for emergency response. A rainfall intensity-duration threshold for the production of debris flows from basins burned by the Coal Seam and Missionary Ridge Fires in the form:

$$I = 0.25D^{-0.5} \qquad (1)$$

where I = rainfall intensity (in in/hr) and D = duration of that intensity (in hours) can be defined (Figures 10A and B).



A.



B.

Figures 10A and B. Rainfall intensity-duration thresholds for the generation of fire-related debris flows from the (A) Coal Seam and (B) Missionary Ridge Fires. Open circles represent measures of storm rainfall from gages near basins that produced debris flows; diamonds represent measures of storm rainfall from gages near basins that produced sediment-laden flows; and dashes represent measures of storm rainfall from gages near basins that showed no response.

Note that the threshold line for the Coal Seam Fire is best defined for durations less than about 2 hrs, while the threshold for the Missionary Ridge fire is fairly well defined for durations up to about 8 hrs. This difference in thresholds could reflect the shorter times to concentration that characterize the generally smaller and steeper debris-flow producing basins of the Coal Seam Fire (Chow et al. 1988). In addition, the rainfall conditions that result in debris-flow activity from recently burned basins are attained at durations at least an order of magnitude less that those described for the generation of debris flows in unburned settings, and at significantly greater intensities. This difference may be attributed to a difference in initiation mechanism. Many workers (e.g., Wells 1981, Parrett 1987, Meyer & Wells 1997, Cannon 2001, Cannon et al. 2001) have found that most debris flows generated from recently burned basins are generated through a process of progressive bulking of storm runoff with materials eroded from hillslopes and channels. In contrast, debris flows in unburned settings are usually found to initiate through the failure of a discrete landslide on the hillslope, which then mobilizes into a debris flow. The difference between the runoff-dominated processes found in burned areas and the infiltration-dominated processes on unburned hillslopes may account for the wide variation in rainfall threshold conditions.

The threshold presented here can provide the basis for warning systems and planning for emergency response in similar settings. That is, for basins that are underlain by similar rock types, have average gradients between 25 and 95 percent and areas less than about 10 mi^2 (25 km^2), are more than 50 percent burned at high and moderate burn severities, and experience summer convective rainstorms.

SUMMARY AND CONCLUSIONS

In this paper we have focused on defining the conditions under which debris flows can occur, and characterizing the magnitude of this response, from the 2002 Coal Seam and Missionary Ridge Fires in Colorado. These are critical elements in post-fire hazard assessments, emergency-response planning, and in the design of mitigation structures.

By documenting the basin area and gradient, and burn extent of the debris-flow producing basins, as well as the rainfall conditions that impacted the basins, we define some of the conditions that lead specifically to the generation of post-wildfire debris flows. Debris flows were produced from basins underlain by interbedded sandstones, siltstones and conglomerates and mantled with SM soils, and from basins underlain by gneissic quartz monzonite and quartzite with SW soils. An abundance of readily entrained materials mantling hillslopes and infilling channels resulted in debris-flow activity throughout the monsoon season. Debris-flow producing basins ranged in size from 0.01 to 8.24 mi^2 (0.03 to 21 km^2), had average gradients between 26 and 94 percent and relief ratios between 16 and 73 percent. A basin area/relief ratio threshold defines the basin morphologic conditions known to have produced post-fire debris flows, given sufficient rainfall. Basins burned at moderate and high severities over more than 50 percent of their areas were susceptible to debris-flow activity. Nearly 70 percent of the debris-flow generating storms were of durations equal to or less than 2 hrs, and 93 percent of these had recurrence intervals of less than or equal to 2 yrs. The average intensities of the debris-flow triggering storms ranged between 0.04 and 0.65 in/hr, with 10-minute peak intensities up to 2.46

in/hr. The conditions described here are likely to produce debris flows from recently burned basins in the future.

Estimates of the peak discharge of debris-flow events are used to define relations between the magnitude of the debris-flow response, storm rainfall triggers, and basin area. Estimates of debris-flow peak discharges between 315 and 5581 ft^3/s (9 and 167 m^3/s) were obtained using indirect methods, and values for peak discharge per unit area ranged between $1.0x10^{-5}$ and $1.2x10^{-3}$ ft/s ($2.0x10^{-6}$ and $3.8x10^{-4}$ m/s). Peak-discharge-per-unit-area values were generally higher for the Coal Seam Fire than for the Missionary Ridge Fire. Debris-flow events with the highest values of peak discharge per unit area, and thus potentially the most destructive, occurred in response to storms with average intensities greater than about 0.4 in/hr (10 mm/hr) and with 10-minute peak intensities greater than about 2.0 in/hr (50 mm/hr).

And last, a rainfall intensity-duration threshold for post-wildfire debris flow activity of the form $I = 0.25D^{-0.5}$, where I = rainfall intensity (in in/hr) and D = the duration of that intensity (in hrs) is defined. Such a threshold is a useful tool in issuing warnings and planning for emergency response for basins underlain by similar materials, of similar sizes and gradients, and burned extents, that experience convective thunderstorms.

Conditions other than those examined here may certainly affect debris-flow occurrence from recently burned basins. Further work is focusing on evaluating the combined effects of a number of variables on debris-flow susceptibility and the magnitude of the response.

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Table 1A. Storm rainfall characteristics and related erosional response of basins within 1 mi (1.6 km) of each rain gage, Coal Seam Fire

Rain gage	Storm Date	Total storm rainfall (inches)	Storm duration (hr:min)	Average storm intensity (in/hr)	Storm recurrence (years)	10-Minute peak intensity (in/hr)	Response of basins located near gage
*Fish Hatchery	8/5/2002	0.48	1:00	0.48	<2	2.28	Debris flow
*Mitchell Creek	8/5/2002	0.31	0:50	0.37	<2	1.20	No response
*South Canyon	8/5/2002	0.67	1:00	0.67	2	1.80	Debris flow
*Storm King	8/5/2002	0.16	1:00	0.16	<2	0.52	No response
**42	8/5/2002	0.35	0:47	0.42	<2	1.92	Debris flow
**43	8/5/2002	0.38	0:50	0.46	<2	2.10	Debris flow
**46	8/5/2002	0.29	0:46	0.39	<2	1.44	Debris flow
*Fish Hatchery	9/7/2002	0.49	2:00	0.24	<2	n/a	Debris flow
*Mitchell Creek	9/7/2002	0.55	2:00	0.28	<2	0.78	No response
**42	9/7/2002	0.30	1:26	0.21	<2	0.60	No response
**43	9/7/2002	0.51	0:47	0.65	2	1.20	No response
**46	9/7/2002	0.32	1:20	0.24	<2	0.54	No response
*Fish Hatchery	9/11/2002	0.16	1:00	0.16	<2	n/a	Debris flow
*Mitchell Creek	9/11/2002	0.25	2:00	0.13	<2	n/a	No response
**42	9/11/2002	0.20	0:13	1.00	<2	1.02	No response
**43	9/11/2002	0.24	0:38	0.38	<2	1.26	No response
**46	9/11/2002	0.08	0:18	0.27	<2	0.42	No response
*Fish Hatchery	9/12/2002	0.09	1:00	0.09	<2	n/a	No response
*Mitchell Creek	9/12/2002	0.27	4:00	0.07	<2	n/a	No response
**Basin C	9/12/2002	0.04	0:20	0.12	<2	0.18	No response
**Basin B	9/12/2002	0.07	0:35	0.12	<2	0.24	No response
**Basin A	9/12/2002	0.12	0:18	0.40	<2	0.54	Sediment-laden flood
*Fish Hatchery	9/17/2002	0.31	2:00	0.16	<2	n/a	Debris flow
*Mitchell Creek	9/17/2002	0.34	2:00	0.17	<2	n/a	No response
**42	9/17/2002	0.22	1:41	0.13	<2	0.36	No response
**43	9/17/2002	0.26	1:18	0.20	<2	0.54	No response
**46	9/17/2002	0.28	1:22	0.20	<2	0.36	No response
*Fish Hatchery	10/2-3/2002	0.99	11:00	0.09	2	n/a	Debris flow
*Mitchell Creek	10/2-3/2002	0.32	6:00	0.05	<2	n/a	No response
**42	10/2-3/2002	0.85	9:24	0.09	2	0.48	Sediment-laden flood
**43	10/2-3/2002	1.09	9:15	0.12	5	0.60	Sediment-laden flood
**46	10/2-3/2002	0.78	10:39	0.07	<2	0.54	No response

*Remote Access Weather Station (RAWS) installed and maintained by U.S.D.A. Forest Service and the Bureau of Land Management.
**Gage installed by USGS

Table 1B. Storm rainfall characteristics and related erosional response of basins within 1 mi (1.6 km) of rain gages, Missionary Ridge Fire

Rain gage	Storm date	Total storm rainfall (inches)	Storm duration (hr:min)	Average storm intensity (in/hr)	Storm recurrence (years)	10-Minute peak intensity (in/hr)	Response of basins located near gage
A3	7/22/02	1.49	8:24	0.18	<2	1.24	Sediment-laden flood
A4	7/22/02	0.46	3:29	0.13	<2	1.08	Ash/Mud flow
A5	7/22/02	1.09	3:14	0.34	2	1.98	Ash/Mud flow
A6	7/22/02	0.16	5:31	0.03	<2	0.18	No response
A8	7/22/02	0.06	2:57	0.02	<2	0.06	No response
A12	7/22/02	0.39	3:15	0.12	<2	0.36	No response
A3	7/23/02	0.13	0:28	0.28	<2	0.42	No response
A4	7/23/02	0.40	2:54	0.13	<2	0.56	Sediment-laden flood
A5	7/23/02	0.50	3:12	0.16	<2	0.84	Debris flow
A6	7/23/02	1.87	1:28	1.25	10	2.40	Debris flow/Flood
A8	7/23/02	0.09	0:30	0.18	<2	0.36	Sediment-laden flood
A12	7/23/02	0.72	1:03	0.69	<2	1.44	Sediment-laden flood
A3	8/3/02	0.12	3:00	0.04	<2	0.24	Debris flow
A4	8/3/02	0.13	0:23	0.34	<2	0.48	Debris flow
A5	8/3/02	0.58	0:54	0.64	<2	2.46	Debris flow
A6	8/3/02	0.04	0:30	0.08	<2	0.18	Debris flow
A6	8/5/02	0.24	0:33	0.44	<2	1.08	Debris flow
A4	8/8/02	0.20	0:21	0.57	<2	0.90	Debris flow
A5	8/8/02	0.18	0:18	0.64	<2	0.84	Debris flow
A10	8/8/02	0.24	1:21	0.18	<2	0.45	Sediment-laden flood
A3	8/20/02	0.33	1:07	0.29	<2	1.14	Sediment-laden flood
A4	8/20/02	0.26	1:33	0.17	<2	0.78	Sediment-laden flood
A5	8/20/02	0.32	1:50	0.17	<2	0.90	Sediment-laden flood
A3	8/21/02	0.13	1:07	0.11	<2	0.24	Sediment-laden flood
A4	8/21/02	0.04	0:12	0.20	<2	0.18	Debris flow/Flood
A5	8/29/02	0.32	0:55	0.35	<2	1.20	No response
A6	8/29/02	0.14	0:43	0.19	<2	0.72	No response
A7	8/29/02	0.09	0:43	0.13	<2	0.24	Debris flow
A10	8/29/02	0.24	0:48	0.30	<2	0.90	Debris flow
A6	9/7/02	0.61	2:15	0.27	<2	0.72	Debris flow
A7	9/7/02	0.17	0:18	0.51	<2	0.60	Debris flow
A8	9/7/02	0.67	2:53	0.24	<2	1.56	Debris flow
A9	9/7/02	0.48	3:02	0.16	<2	1.38	Debris flow
A10	9/7/02	0.60	2:50	0.21	<2	1.11	Debris flow
A12	9/7/02	1.11	2:15	0.49	2	3.11	Sediment-laden flood

Table 1 B, continued.

Rain gage	Storm date	Total storm rainfall (inches)	Storm duration (hr:min)	Average storm intensity (in/hr)	Storm recurrence (years)	10-Minute peak intensity (in/hr)	Response of basins located near gage
A4	9/10/02	0.30	1:42	0.18	<2	0.54	Sediment-laden flood
A5	9/10/02	0.81	1:33	0.54	<2	1.68	Sediment-laden flood
A4	9/11/02	1.44	13:15	0.11	<2	0.30	Debris flow
A6	9/11/02	0.22	1:11	0.16	<2	0.72	Debris flow
A7	9/11/02	0.16	0:59	0.16	<2	0.72	Debris flow
A8	9/11/02	1.84	12:28	0.15	2	0.72	Debris flow
A9	9/11/02	0.89	12:40	0.07	<2	0.24	Debris flow
A10	9/11/02	1.82	14:18	0.13	2	0.60	Debris flow
A7, A10	9/20/02	--	--	--	--	--	Debris flow (no rainfall recorded)
A8	10/2/02	0.78	13:00	0.06	<2	0.27	Sediment-laden flood
A9	10/2/02	0.10	1:15	0.08	<2	0.12	Sediment-laden flood

Table 2A. Peak discharge estimates and peak discharge estimates normalized by basin area for debris flows generated from basins burned by Coal Seam Fire

Basin	Event date	Average cross-sectional area (ft^2)	Peak discharge (ft^3/s)	Peak discharge/basin area (ft/s)
A	8/05/02	30.8	493*	0.00002
B	8/05/02	48.8	781*	0.00012
C	8/05/02	19.7	315*	0.00019
D	8/05/02	24.4	390*	0.00070
F	8/05/02	63.0	630**	0.00113
G	8/05/02	40.6	406**	0.00730
H	8/05/02	34.7	347**	0.00124
M	8/05/02	21.8	349*	0.00021

*Calculated as Q = VA, with average velocity assumed to be 16 ft/s (5 m/s)
**Calculated as Q = VA, with average velocity assumed to be 10 ft/s (3 m/s)

Table 2B. Peak discharge estimates and peak discharge estimates normalized by basin area for debris flows generated from basins burned by Missionary Ridge Fire.

Basin	Event date	Average cross-sectional area (ft^2)	Peak discharge (ft^3/s)*	Peak discharge/basin area (ft/s)
Root	8/8/02	95.1	1522	0.00007
Root	8/21/02	109.4	1750	0.00008
Unnamed 2	8/8/02	46.8	748	0.00110
Haflin	8/29/02	18.3	293	0.00001
Freed	9/7/02	328.3	5253	0.00008
Stevens	9/7/02	348.8	5581	0.00003
Kroeger	8/29/02	47.3	757	0.00002
Coon	9/7/02	93.4	1494	0.00001
Woodard	9/7/02	93.4	1494	0.00006
Unnamed 3	8/8/02	83.5	1336	0.00015
Unnamed 1	8/5/02	57.5	920	0.00006
Mayers	9/5/02	54.8	877	0.00005
Mayers	9/10-12/02	37.6	602	0.00004

*Calculated as Q = VA, with average velocity assumed to be 16 ft/s (5 m/s)

Table 3A. Characteristics of debris-flow producing basins within the Coal Seam Fire.

Basin	Area (mi^2)	Average gradient (%)	Relief ratio (%)	Geologic unit	Percent basin unburned	Percent basin burned at low severity	Percent basin burned at moderate severity	Percent basin burned at high severity	Percent basin burned at moderate and high severities
A	0.81	51	24	1	0	1	38	61	99
B	0.22	53	33	1	17	3	46	35	81
C	0.06	55	40	1	6	3	62	29	91
D	0.02	49	39	1	2	10	45	43	98
E	0.05	46	39	2	0	8	57	25	82
F	0.02	74	64	2	0	1	96	3	99
G	0.02	71	67	2	0	0	89	11	100
H	0.01	69	65	2	0	0	96	4	100
	0.03	64	61	2	0	1	80	19	99
	0.05	84	73	1	0	0	17	83	100
K	0.57	52	30	1	42	4	8	46	54
L	0.14	60	54	1	2	11	46	41	87
M	0.06	87	63	1	6	13	26	55	81
N	0.07	88	64	1	3	12	26	58	84
O	0.06	94	60	1	12	25	19	44	63

1. Maroon Formation: interbedded sandstones, shales, siltstones and conglomerates (Kirkham et al., 1997)
2. Sawatch Quartzite and Gneissic quartz monzonite of Mitchell Creek (Kirkham et al, 1997)

Table 3B. Characteristics of debris-flow producing basins within the Missionary Ridge Fire.

Basin	Area (mi^2)	Average gradient (%)	Relief ratio (%)	Geologic units	Percent basin unburned	Percent basin burned at low severity	Percent basin burned at moderate severity	Percent basin burned at high severity	Percent basin burned at moderate and high severities
Root	0.73	32	17	1	0	1	29	69	98
Unnamed 2	0.24	39	25	1	0	6	56	39	95
Haflin	1.60	56	23	1, 2	4	11	34	51	85
Freed	2.18	53	18	1, 2	1	4	43	53	96
Stevens	6.08	52	14	1, 2, 3	2	12	50	35	85
Kroeger	1.21	53	25	1, 2	1	9	65	25	90
Coon	8.04	37	16	1, 2, 3	4	18	33	45	78
Woodard	0.92	58	30	1, 2	8	19	51	22	73
Unnamed 3	0.33	30	24	2	0	14	57	28	85
Unnamed 1	0.57	26	21	4	0	6	46	48	94

1. Cutler Formation: interbedded sandstones, shales, siltstones and conglomerates
2. Junction Creek, Wanakah, Entrada, and Dolores Formations: sandstone, with some shale and siltstone
3. Hermosa and Molas Formations: shale, limestone and sandstone
4. Morrison Formation: claystone, siltstone mudstone and sandstone

HISTORICAL DEBRIS FLOWS ALONG THE INTERSTATE-70 CORRIDOR IN CLEAR CREEK COUNTY, CENTRAL COLORADO

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Key Terms: debris flow, Interstate 70, Clear Creek County, Idaho Springs, Georgetown, Brownville, rainfall, monsoon, snowmelt

ABSTRACT

We have compiled information on 19 historical debris-flow events along the Interstate-70 corridor in Clear Creek County. Twelve of these events were triggered by rainfall, and seven by snowmelt. Of the twelve triggered by rainfall, ten were caused by rainstorms during July and August. At least five of the seven snowmelt-triggered events involved failures of mine dumps that were located in drainage channels. Debris flows were most common on steep, south-facing hillslopes above the maximum extent of Pleistocene glaciation. Observations of recent debris flows suggest that most flows were initiated by erosive processes of rilling and/or a “fire-hose” effect, in which overland flow that is concentrated in bedrock-lined channels impacts and mobilizes debris from talus deposits and the heads of debris fans.

INTRODUCTION

The Interstate-70 (I-70) corridor in Clear Creek County (Figure 1) has been repeatedly identified by the Colorado Geological Survey (CGS) as one of the most serious landslide hazard areas in Colorado (Colorado Water Conservation Board, 1985; Jochim et al., 1988; Rogers, 2003). The CGS uses the term “landslide” to include all types of slope failures, including debris flows. The I-70 corridor in Clear Creek County is listed by the CGS as a debris-flow and rockfall hazard area. In this area, I-70 transects the Front Range along Clear Creek, an eastward-flowing, formerly glaciated drainage. Debris flows initiate in tributary drainages of Clear Creek and can transport debris from above timberline to multiple lanes of I-70. This paper documents historical debris flows that have occurred along the I-70 corridor in Clear Creek County. We include debris flows that occurred within the corridor prior to the construction of I-70 in the 1960s and 1970s. Our hope is that the information contained in this paper will contribute to an understanding of debris-flow triggers, processes, frequencies, and hazards in Clear Creek County.

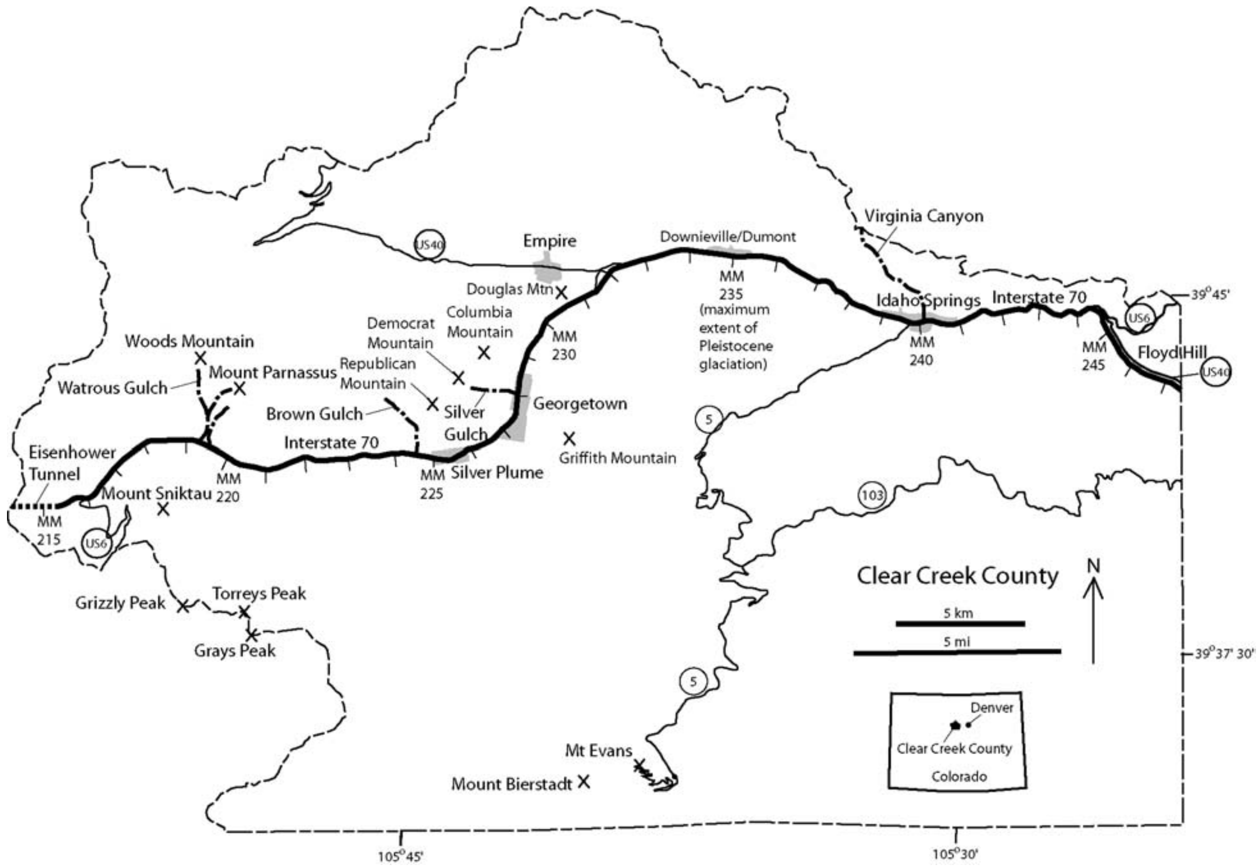

Figure 1. Map showing location of Clear Creek County and Interstate 70. The area is within the north-south trending Front Range, the eastern-most mountain range in Colorado. Shaded areas are towns. Road and highway numbers are labeled. Mile markers (MM) are shown as tick marks along I-70.

SETTING

Physiographic

The Clear Creek valley in Clear Creek County is located from about 19 mi (30 km) to 47 mi (75 km) west of Denver. The upper part of the Clear Creek valley (elevations above about 7950 ft (2420 m)) was repeatedly glaciated during the Pleistocene (Madole et al., 1998). The most recent Pleistocene glaciers (Pinedale age) in the Clear Creek valley are estimated to have disappeared between 14,000 and 12,000 ^{14}C yr BP (Caine, 1986, Madole et al., 1998). The maximum extent of Pleistocene glaciation is located near Dumont (elevation of about 7950 ft (2420 m), Figure 1, Madole et al., 1998). Above this boundary, the Clear Creek valley typically has steep walls and small, steep tributary drainage basins. Most drainages contain glacial deposits, and talus deposits are common at the foot of steep bedrock hillslopes. The lower, non-glaciated part of the Clear Creek valley is generally characterized by moderately steep hillslopes and large, moderately steep tributary drainage basins. Below the glacial limit, Pleistocene gravels are present along the valley bottom and on hillslopes adjacent to Clear Creek. Hillslopes in both the glaciated and non-glaciated parts of the area are commonly mantled by matrix-supported colluvium. Fans are present at the mouths of tributary drainage basins in both parts of the valley.

The area is underlain predominantly by Precambrian biotitic gneiss and quartz monzonite with scattered Tertiary intrusions (Spurr et al., 1908; Lovering, 1935; Sims, 1964; Braddock, 1969; Sheridan & Marsh, 1976; Bryant et al., 1981; Widmann et al., 2000; Widmann & Miersemann, 2001) with associated hydrothermal alteration and silver-and-gold mineralization (Harrison & Wells, 1956; Sims & Gable, 1967). The zone of mineralization that encompasses the area extends from southwestern Colorado to the Front Range northwest of Denver, and is known as the Colorado Mineral Belt (Tweto & Sims, 1963). Mining activity was common in the area in the late 1800s and early 1900s and numerous abandoned mines and mine dumps are present on hillslopes in the area.

Elevations within the Clear Creek valley range from about 7220 ft (2200 m) at Floyd Hill to about 10,990 ft (3,350 m) at the east entrance to the Eisenhower tunnel. Mountain peaks adjacent to the Clear Creek valley range up to about 14,270 ft (4350 m) in elevation. Mean annual precipitation ranges from about 15 in (380 mm) in Idaho Springs (elevation 8150 ft (2,484 m)) to about 33 in (840 mm) near Grizzly Peak (elevation 11,949 ft (3642 m)) near the Arapahoe Basin ski area (Western Regional Climate Center, 2003, unpublished data).

Tree cover in the area ranges from a Ponderosa Pine, Juniper, Douglas Fir assemblage at lower elevations, to an Englemann Spruce, Limber Pine, Subalpine Fir assemblage at higher elevations. Timberline is at an elevation of about 11,480 ft (3,500 m). Above timberline, hillslopes are bare or are covered by alpine tundra. In general, hillslopes on the south side of I-70 (north facing) have more vegetation than hillslopes on the north side (south facing) of I-70, presumably because the difference in solar exposure results in soil conditions that are cooler and wetter on the south side, and warmer and drier on the north side.

Interstate 70

Interstate 70 (I-70) in Colorado is the main east-west highway route serving the Denver metropolitan area, one of the fastest growing regions of the United States. Increasing traffic associated with the growth in population has led to traffic congestion on I-70 east of the Continental Divide, along the mountainous Front Range portion of the highway that parallels Clear Creek (Figure 1). Desire to alleviate this congestion has motivated recent investigations into modifications of transportation infrastructure that would increase the capacity along the Front Range portion of the I-70 corridor (Andrew & Lovekin, 2002; Arndt et al., 2002). Modifications that have been proposed include additional highway lanes, a monorail, and an additional highway tunnel under the Continental Divide (there are currently two individual tunnels that are jointly referred to as the Eisenhower Tunnel). Assessments of geologic hazards provide critical baseline information that can be used to evaluate the proposed infrastructure modifications within the corridor (Andrew & Lovekin, 2002).

Previous work on debris flows along Interstate 70

Recent debris flows, as well as Holocene debris-flow deposits, have shown that the Front Range part of the I-70 corridor is susceptible to debris-flow hazards (Soule, 1975; Hecox, 1977; Pelizza, 1978; Coe & Godt, 1997; Coe et al., 1998; Soule, 1999; Henceroth, 2000; Coe et al. 2002; Godt & Coe, 2003; Andrew & Lovekin, 2002; Widmann & Rogers, 2003). Debris flows along I-70 in Clear Creek County initiate on hillslopes in tributary drainage basins of Clear Creek and form fans at the mouths of the basins along the north and south flanks of the Clear Creek valley. An analysis of historical and stratigraphic records of debris-flow events at 19 fans in the corridor resulted in estimates of mean recurrence intervals (the average time between debris-flow events) at the fans (Coe et al., in press). Mean recurrence intervals ranged from about 7 to 2900 yrs. Field observations made during the same study indicated that mean recurrence intervals tended to be shortest on fans at the mouths of small and steep basins and longest on fans at the mouths of large basins with low-to-moderate relief. Following these observations, a method was developed (Coe et al., in press) to estimate the probability of future debris flows on fans along Clear Creek using a measure of drainage-basin ruggedness, called Melton's Number. Melton's Number is unitless and is defined as $H/(A)^{0.5}$, where H is basin height upstream from the fan and A is basin area upstream from the fan (Melton, 1965). Melton's Numbers can be easily derived from a Digital Elevation Model (DEM). Basins that are small and steep have higher Melton's Numbers than larger basins with low-to-moderate relief. A regression analysis of mean recurrence intervals and Melton's Numbers from the 19 fans and corresponding basins yielded the equation $y=19{,}400\exp^{-4.67x}$, where y is mean recurrence interval in years and x is Melton's Number (Figure 2). Following verification through further work (see Coe et al., in press), it may be possible to use this equation to estimate the mean recurrence interval for debris flows on fans with no historical or stratigraphic records from the Melton's Numbers of the corresponding drainage basins.

Observations of debris flows along I-70 that we have made since the summer of 1996 (described in Coe et al., 2002; Godt & Coe, 2003; and Coe et al., in press) suggest that one of the reasons that debris flows occur frequently on fans at the mouths of basins with relatively large Melton's Numbers is that they have a greater likelihood of flowing to the fan than do debris flows in basins with relatively small Melton's Numbers. We suspect that if debris flows were to occur with equal frequency on hillslopes in all basins, many of the debris flows in the large basins with low to moderate relief (small Melton's Numbers) would deposit material at the base of hillslopes within the basins, not on fans at the mouths of basins. This would also explain why fans at basins with very small Melton's Numbers are dominated by flood events (Coe et al., in press), rather than debris-flow events. In basins with high Melton's Numbers, the base of the hillslope and the mouth of the basin are essentially the same. Debris flows in these basins simply flow down the hillslopes and are deposited on fans.

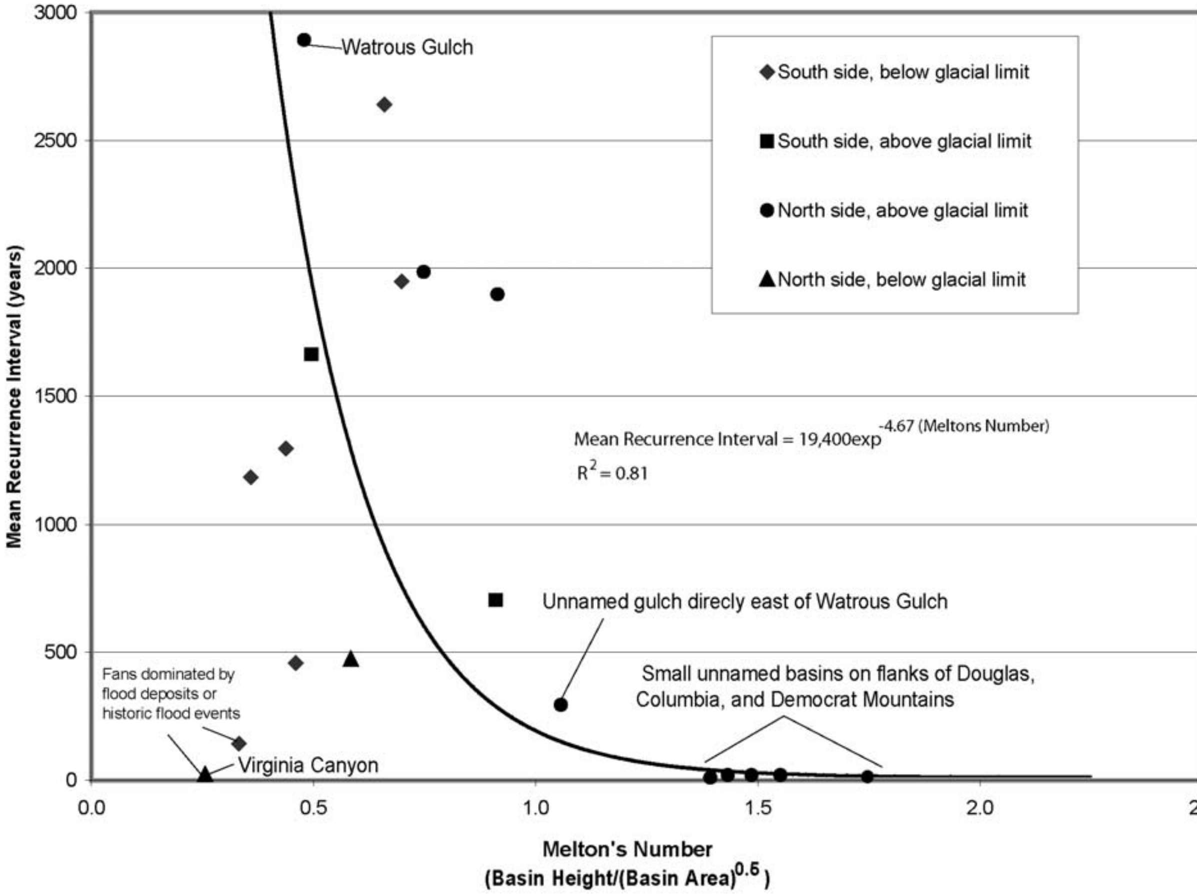

Figure 2. Scatter plot showing mean recurrence interval data from fans, and Melton's Number data from corresponding basins along I-70 in Clear Creek County (from Coe et al., in press). Mean recurrence interval data were derived from stratigraphic and/or historical data at each fan. A regression analysis yielded the best-fit line and equation. Fans dominated by flood deposits or historical flood events were not used in the regression analysis. Fans/basins with historical debris-flow events discussed in the text are labeled. Location of fans on the north or south side of I-70, and above or below the glacial limit, is also shown.

METHODS

For this paper, we compiled information on historical debris flows using newspaper articles, published reports, eyewitness accounts, and personal observations. We searched for newspaper articles that described slope failures using the keywords "avalanche", "mudslide", "earth movement", "landslide", "rockslide", and "flood". If an article described a generally fast-moving slope failure and that carried debris (mud, rocks, vegetation, etc.), we classified the failure as a debris flow. Newspaper articles in the *Rocky Mountain News* and *Denver Post* were reviewed using on-line, hard copy (annual book), and card catalog versions of subject indices available through the Denver Public Library. Publication of the *Rocky Mountain News* and *Denver Post* began in 1859 and 1895, respectively. The *Rocky Mountain News* is indexed in on-line form from 1989 to present, whereas the *Denver Post* is indexed in either book or on-line form from 1979 to present. Card catalog subject indices available at the library were sporadically created throughout the 1990s, and are therefore incomplete. Microfilm versions of both newspapers are available from the initial publication date to the present. We attempted a comprehensive search of each newspaper by visually scanning microfilm tapes and found that it took about 1 hr to review 20 days of daily newspapers (two microfilm tapes). At this rate, it would take about 1800 hrs to review 100 yrs of daily newspaper articles. For this reason, we have not done a comprehensive review of newspaper articles available on microfilm.

Transcripts of landslide-related articles from Clear Creek County newspapers from the late 1880s through the early 1900s were provided by Christine Bradley, the Clear Creek County Archivist. She provided articles from the *Colorado Miner* and *Georgetown Courier* (both Georgetown newspapers) and the *Silver Standard* (a Silver Plume newspaper).

The record of historical debris flows documented in this paper is incomplete because newspaper articles prior to 1979 were not systematically indexed or searched, and because there have undoubtedly been debris flows that were not observed or recorded in any report or newspaper.

HISTORICAL DEBRIS FLOWS

We have documented 19 historical debris-flow events that occurred adjacent to the location of I-70 in Clear Creek County (Table 1). We define a debris-flow event as an occurrence of debris flow(s) in one or more basins. Fourteen of the debris-flow events initiated in tributary basins above the glacial limit on the north side of the highway (Figure 1). There were two types of triggers for debris flows along I-70: rainfall (12 events) and snowmelt (seven events). Ten of the 19 debris-flow events were triggered by rainstorms during July and August, storms that are commonly associated with the northerly flow of monsoon moisture from the Gulf of California and Eastern Pacific Ocean (referred to as the North American Monsoon by Adams & Comrie (1997)). This pattern has also been observed in the San Juan Mountains of southwestern Colorado, where more than 90 percent of historical debris flows have been triggered by rainstorms in the months of July and August (Coe & Burke, 2003). At least five of the seven snowmelt-triggered debris flows were related to failures of mine dumps located in channels.

There were debris flows that we identified during our compilation that were in Clear Creek County, but not directly along I-70, and therefore not listed in Table 1. These debris flows included several hundred that were triggered by a rainstorm on July 28, 1999 (see Godt and Coe, 2003, for map and description), as well as two more that occurred on July 16, 2000, and resulted in closures of Colorado Highways 5 and 103 near Mount Evans (Vaughan & Kass, 2000).

Our compilation also revealed information on several historical deep-seated landslides in Clear Creek County. Although we did not systematically compile these landslides for this paper, several are worthy of mention. These include the Floyd Hill landslide (Rocky Mountain News, 1947; Robinson et al., 1974), the Loveland Basin landslide (Robinson & Lee, 1972), the Clear Creek Forks landslide (Savage et al., 1998), and a landslide near the Lower Cabin Creek Dam south of Georgetown that killed three people in 1965 (Myers, 1965).

Below, we describe the historical debris-flows locations that have had repeated events or single events that have greatly impacted the I-70 corridor. We begin with locations near the eastern edge of the County and proceed to the west.

Idaho Springs and vicinity

The historical debris-flow event located farthest east in Clear Creek County is on the north flank of Floyd Hill near mile marker 245 (Figures 1 and 3, Event 1 in Table 1). To our knowledge, this was the only debris flow triggered by a rainstorm in late July 2000. This debris flow initiated as a shallow landslide (also referred to as a soil slip, Campbell, 1975; also see Reid et al., 1988 and Iverson et al., 1997) in fill material beneath I-70 (Figure 3). Another debris flow occurred at Floyd Hill on July 8, 2001 (Event 2, Table 1). This debris flow deposited debris on the eastbound lane(s) of the highway.

Figure 3. Debris flow along the north flank of I-70 at Floyd Hill. View is to the south. Photo taken August 4, 2000. See guard rails along I-70 and US Highway 40 for scale.

Small drainage basins on the north side of Idaho Springs, as well as the basins directly west of town along I-70, have had historical debris-flow occurrences (Events 3 and 4 in Table 1). These basins tend to be small and steep.

A much larger drainage basin near Idaho Springs that is recognized as a hazardous area is Virginia Canyon (Figure 1). Hillslopes in Virginia Canyon have a long history of mining activity (Stewart & Severson, 1994), and the Virginia Canyon channel contains an abundance of debris, some of which originated from mine dumps. The mouth of Virginia Canyon in Idaho Springs has a history of water-dominated (flood) or hyperconcentrated flow (Pierson and Costa, 1987) events, but to our knowledge, not debris-flow events. Herron et al. (2001) reported that *"… in recent years that there have been several thunderstorms that have resulted in flooding of the housing area near the confluence …"* of Virginia Canyon and Clear Creek. There have been water-dominated or hyperconcentrated flows in the Canyon and on the fan in August 1994, possibly in July 1997, and in July 1998. We have visited the fan and canyon multiple times and have observed deposits following the 1998 event. The hazardous area during flow events is along the channel near the mouth of the canyon. During such events, water and debris run down the channel, as well as on the road. This presumably occurs because culvert pipes get clogged with debris. Houses near the channel are also impacted by these events. One structure is built on top of the channel and others have structural components anchored within the channel. Once flows reach the fan, they are contained within a concrete-lined channel.

Debris flows on the flanks of Douglas, Columbia, and Democrat Mountains

The most active debris-flow area along I-70 is on the north side of the highway between the US Highway 40/I-70 junction and Georgetown along the southern and eastern flanks of Douglas, Columbia, and Democrat Mountains (Figures 1, 4-6, Events 5-11 in Table 1). The valley in this area has been glaciated and is U-shaped with small and steep tributary basins (Figure 4). Tributary basins in this area tend to have Melton's Numbers greater than 1, are sparsely vegetated, and are dominated by bedrock slopes in their upper portions and fan aprons in their lower portions. The mean recurrence interval between recent debris-flow events in this area is about 7 yrs or less (Coe et al., in press).

Debris flows in this area tend to initiate from hillslope and channel erosion, not from discrete landslide source areas as documented in many other parts of the United States. Two processes are responsible for mobilizing debris, a "fire-hose" process (Johnson & Rodine, 1984; Fryxell & Horberg, 1943; and Curry, 1966), and a progressive rilling process. The fire-hose process begins when flowing water in steep bedrock channels crosses talus or debris-fan material at the base of steep bedrock slopes. When the concentrated water-dominated flows impact these materials, they erode and mobilize debris. Progressive rilling is the concentration of flow that mobilizes loose sediment primarily at knickpoints and plunge pools (Horton, 1945; Johnson & Rodine, 1984; Cannon et al., in press). Debris flows that initiate by these processes in this area tend to travel short distances (less than 0.6 mi (1 km)) and deposit volumes of material less than 1300 yd^3 (1000 m^3).

Figure 4. Oblique aerial photo showing historical debris-flow areas near Georgetown. U.S. Geological Survey photograph taken on August 18, 1999 by Intrasearch, Inc. View is to the north.

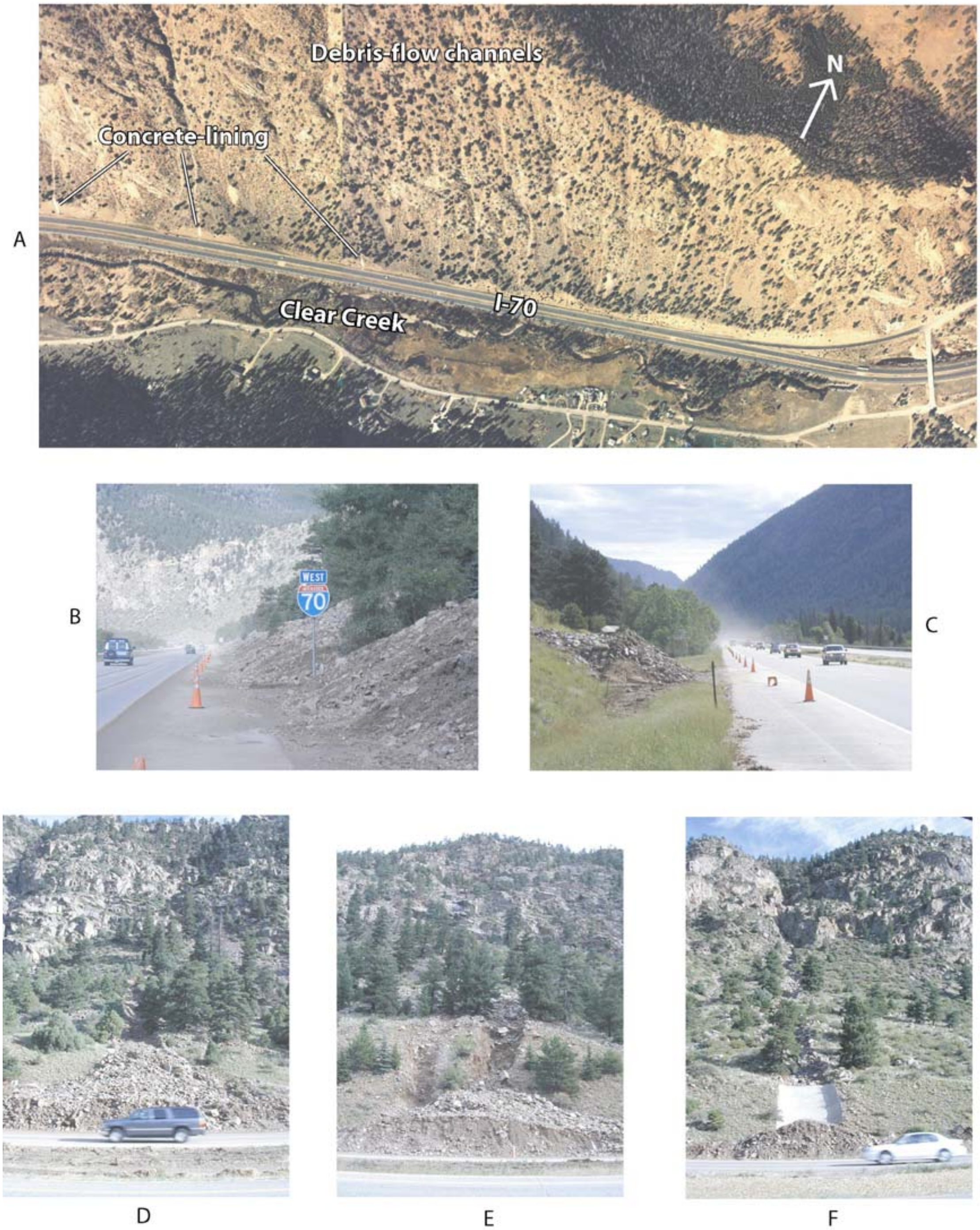

Figure 5. Debris flows along I-70 on the south flank of Douglas Mountain near the junction of I-70 and U.S. Highway 40. A) Aerial photograph of the area. Photograph taken October 12, 1996. B) Piles of debris from flows on July 13, 2001. View to southwest. C) Deposit and dust from July 13 debris flows. View to northeast. D,E,F) July 13, 2001, debris-flow channels and deposits. View to northwest. Concrete-lined channel shown in F. Photographs B through F were taken on July 14, 2001.

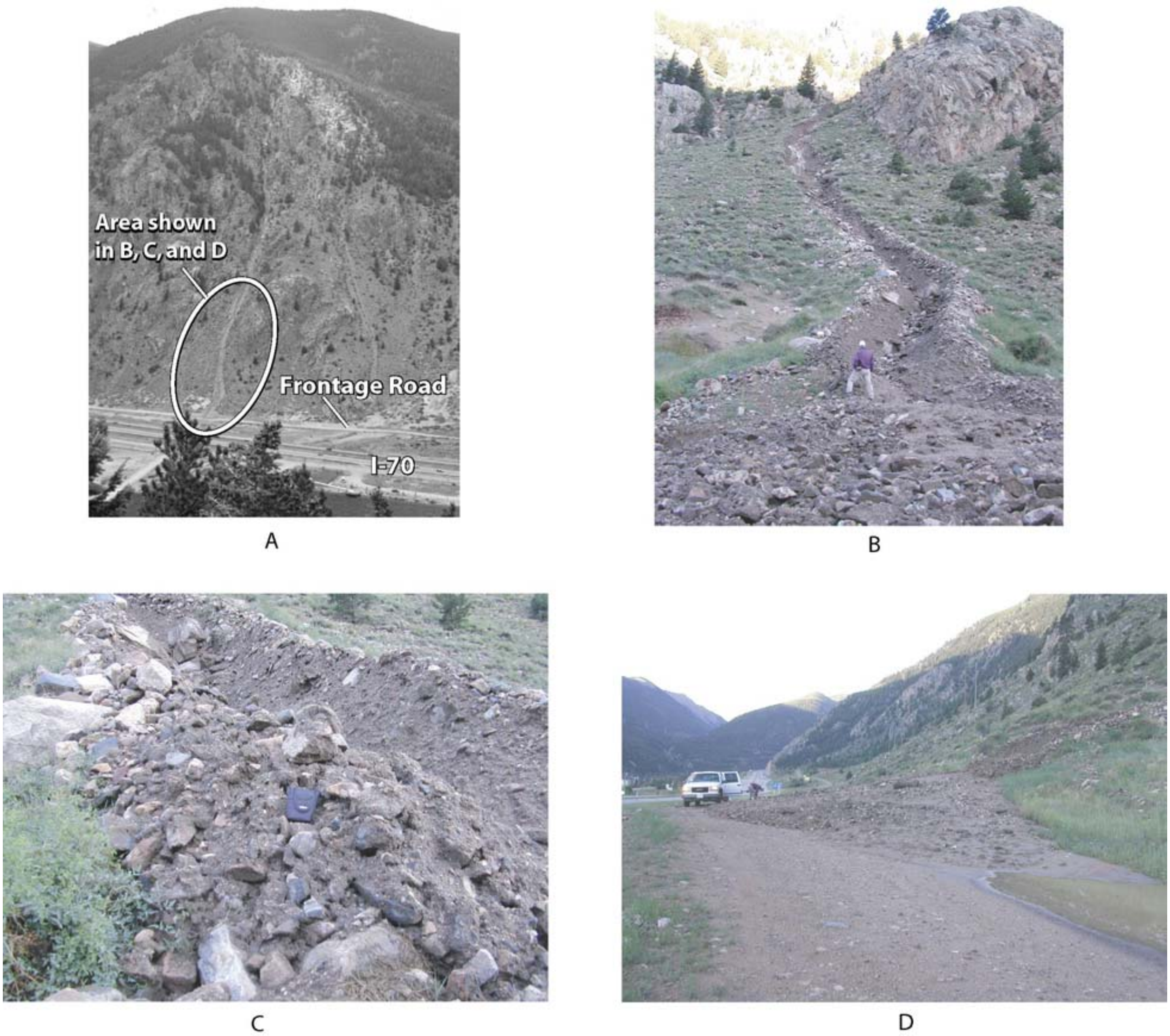

Figure 6. Drainage basin and July 13, 2001, debris-flow deposits near Georgetown (see Figure 4 for location). A) Drainage basin. Photograph taken August 30, 1996. B) Debris-flow channel. C) Matrix-supported debris-flow levee. Note camera case for scale. D) Fan formed by deposit. Photos B, C, and D were taken July 14, 2001.

A recent debris-flow event that affected this portion of I-70 was triggered by an afternoon rainstorm on July 13, 2001 (Event 7 in Table 1). The storm triggered about 13 debris flows that deposited debris on the west-bound lane of I-70 (Figure 5) between mile markers 230.5 and 231.5. Debris on westbound lanes was as much as about 10 ft (3 m) thick and extended to the highway centerline. The Colorado Department of Transportation (CDOT) closed the Interstate between the Empire and Georgetown exits for about 4 hrs while debris was cleared from one westbound lane. Much of the mobilized debris came from the oversteepened, cut-slope portion of the talus fan directly adjacent to I-70 (Figure 5e). In order to minimize the amount of material mobilzed from the cut-slope part of the talus fans, CDOT had previously installed concrete linings in cut-slope channels along the most active debris-flow paths (Figures 5a and 5f).

Observations of the amount of material deposited on I-70 during the July 13 storm suggest that these concrete linings are effective in reducing the amount of debris eroded and mobilized from the cut slope during rainstorms (compare Figures 5e and 5f).

One of the best locations to observe July 2001 debris-flow features in the area is on the north side of I-70 near Georgetown (Figure 6). The fan at this location is matrix-supported, poorly sorted, and contains randomly oriented clasts. Grain-size analysis (U.S.Geological Survey, 1999, unpublished data) of a similar deposit at the same location that resulted from a July 27 or 28, 1999, rainstorm (Event 11, Table 1) indicated that sand-sized material makes up most of the matrix (74 percent), followed by silt- (21 percent), and clay-sized material (5 percent). Unlike some of the other debris-flow channels in the area, this channel has matrix-supported levees that seemingly receive fresh material with each debris-flow event (e.g., see Figure 6c).

Grain-size data throughout this paper are presented on the basis of an engineering soil classification (see Terzaghi and others, 1996) with size distinctions as follows: sand: 4.76 mm to 0.074 mm; silt: 0.074 mm to 0.002 mm; and clay: less than 0.002 mm. We use the term *matrix* when referring to the sand- through clay-sized portion of material in deposits.

Silver Gulch

A type of debris flow that differs from those described above was triggered by spring snowmelt in Silver Gulch (Figures 1 and 4) in May 1872 (Event 9 in Table 1). Silver Gulch is an atypical basin for the area in that it is relatively large (Melton's Number: 0.76). On May 28, 1872, snowmelt and runoff from streams and springs triggered a landslide along the bank of Silver Creek near the Beecher Silver Mine. The landslide dammed Silver Creek about noon on May 28. The landslide dam failed and the rushing water carried wood, logs, and debris downstream where it formed another dam estimated to be about 40 ft (12 m) high. This second dam failed about 12:30 pm and the rushing water mobilized debris and carried it to the fan at the mouth of the basin where it damaged several houses and deposited debris as much as about 6 ft (2 m) deep.

Brown Gulch

Brown Gulch (Figures 1, 7, and 8, Melton's Number: 0.53) was the site of at least 5 debris-flow events between 1889 and 1912 (table 1, Events 14-18). The primary source of debris in the flows was mine-waste dumps located within the Gulch. Most of the mobilized debris came from the waste dump of the Seven-Thirty mine (Figure 7) located at an elevation of about 10,450 ft (3,185 m), about 1200 ft (365 m) upstream from the fan (Figure 7) at the mouth of the Gulch. At least one of the debris-flow events buried buildings in the small town of Brownville, located on and near the fan. All of the documented debris-flow events occurred in the month of June and were apparently triggered by snowmelt. Part of a *Silver Standard* (1892) article from June 25, 1892, mentions the " first really warm day" of spring followed by a "rapid rise of water" in creeks. Part of the discussion of Brown Gulch is quoted as follows.

> "*On Wednesday morning the dump of the Seven-Thirty mine began to wash away and Brown Gulch was a scene of ruin. The water in the gulch seemed to reach its*

highest point after midnight and when the first dump started to go the mass of rock and timbers was added to from other dumps as it went down. The ore houses and blacksmith shops of the Coin, Brown, Mammoth and Dunderberg were situated in the gulch, as are also the mouths of numerous tunnels going into the mines. The latter were speedily covered over and the buildings either buried where they stood or washed down the gulch. Most of the debris washed down on Wednesday stopped at the lower end of the gulch above the Terrible, and on Thursday morning about 3 o'clock it began to move under the influence of the volume of water then coming down. It was expected that it would go toward the Union tunnel of the Terrible but it took a course toward the Terrible mill and granite quarry, burying the house occupied by William Payne and the office of the company. The mass of rock flowed out over the railroad track running into the quarry and filled up the wagon roads going across the bridge and by the Terrible Mill to a depth of many feet. One corner of the mill is mashing in, and 2 cars of rock standing on the track were buried."

An examination of the Seven-Thirty mine dump in October 2002 (Figure 8) revealed that a large part of the dump is still present (Figures 8a and 8b) and that it lies adjacent to the channel (Figure 8c). Presumably, when the debris flows occurred in the late 1800s and early 1900s, the dump was closer to, or covered, part of the channel. Thus, the high volume of runoff from snowmelt eroded the edge of the dump and caused it to fail into the channel and contribute debris to the runoff. The dump could still contribute debris to the channel if it failed as a landslide; however, because the dump is now located farther from the edge of the active channel, it appears that "normal" spring runoff would no longer erode the edge of the dump and be the cause for such a landslide failure. Modern debris flows in Brown Gulch would most likely be triggered by intense or prolonged rainfall.

Mount Parnassus and vicinity

On July 28, 1999, about 480 alpine debris flows were triggered by an afternoon rainstorm along and near the Continental Divide in Clear Creek and Summit Counties (Godt & Coe, 2003). The rainstorm dropped 1.7 in (43 mm) of rain in 4 hrs, most of which (1.4 in (35 mm)) fell in the first 2 hrs (Event 19, Table 1). Field observations of debris-flow source areas indicate that the debris flows were initiated by three processes, fire hose, coalescing rills, and soil slips, with the first two processes being responsible for most of the debris flows (Godt and Coe, 2003).

Several debris flows triggered by the storm affected I-70, U.S. Highway 6, and the Arapahoe Basin ski area. Several debris flows initiated on the south flank of Mount Parnassus (Figures 1 and 9, Event 19 in table 1), traveled about 1.5 mi (2.5 km) down Watrous Gulch and an unnamed gulch directly to the east of Watrous Gulch, and deposited about 34,000 yds (26,000 m^3) of debris on I-70 (Al Chleborad, 1999, written communication), closing the highway for about 25 hrs. Fortunately, little permanent damage to public or private property and no injuries or fatalities resulted from any of the flows.

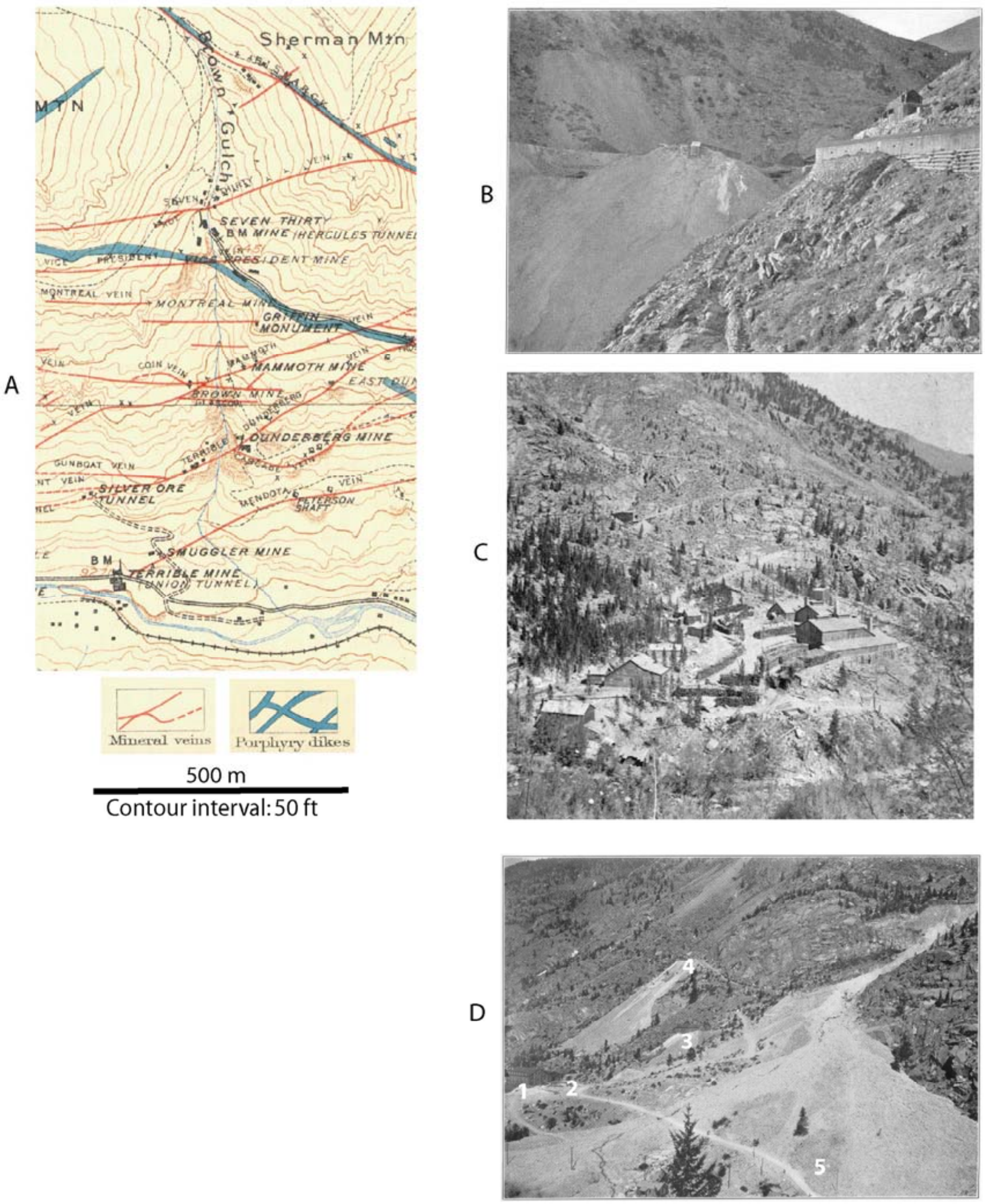

Figure 7. Map and photos of Brown Gulch in late 1800s and early 1900s (A, B, and D from Spurr et al. (1908); C from Denver Public Library Historical Photograph Collection, number X-7236). A) Map made in 1906 showing topography, mines, and veins and dikes in Brown Gulch and vicinity. B) Seven-Thirty mine and dump in Brown Gulch looking northwest from the Griffin monument. C) Fan at the mouth of Brown Gulch taken between 1886 and 1898. View to the northeast. Compare to Figures 7D and 8F. D) Fan at the mouth of Brown Gulch showing; 1, Terrible mill; 2, Union tunnel of the Terrible mine; 3, Smuggler mine; 4, Silver Ore tunnel of the Terrible Mine; 5, deposit of debris mobilized from mine dumps in Brown Gulch.

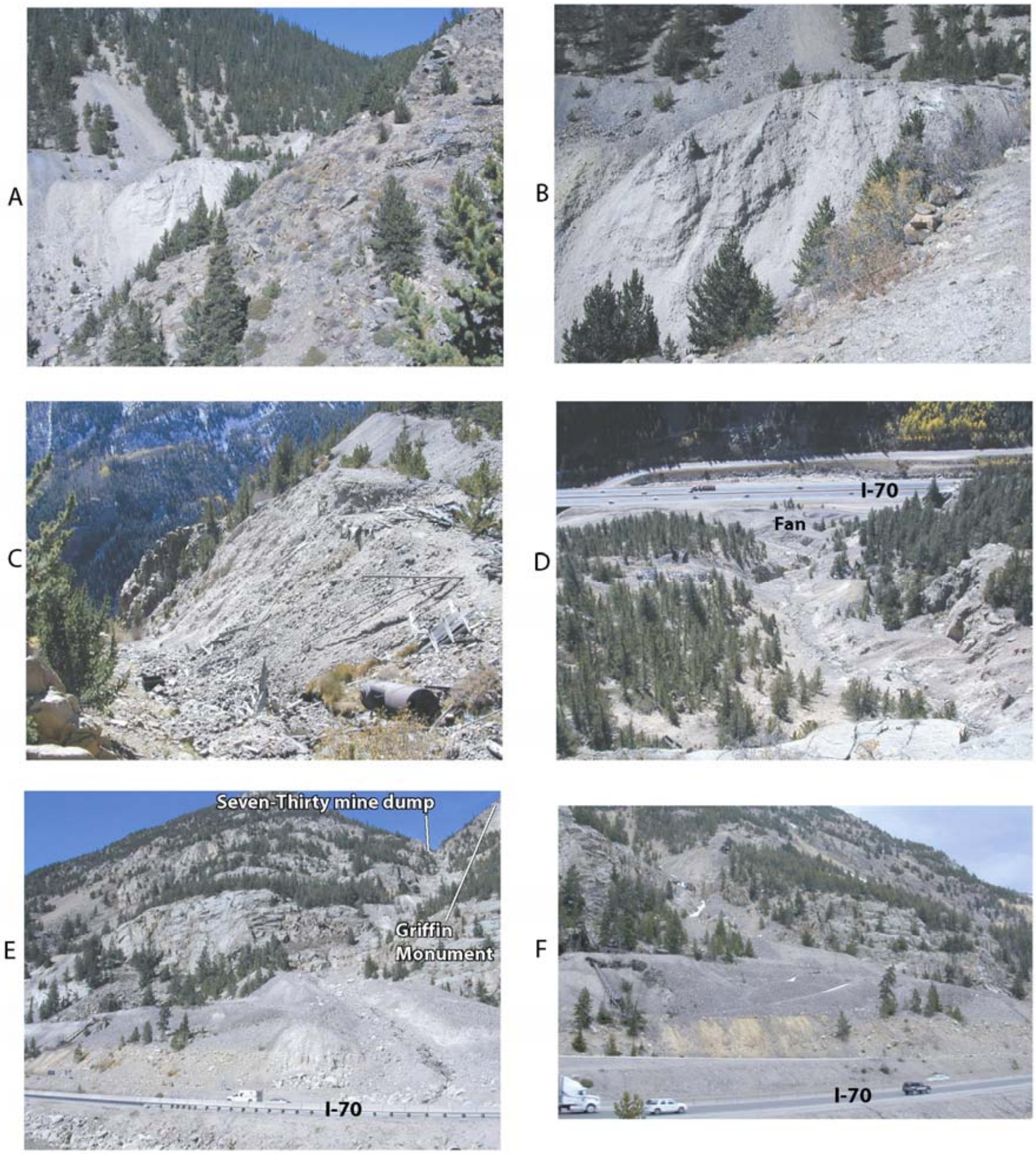

Figure 8. Brown Gulch in 2002 and 2003. Photos A-E taken on October 10, 2002. A) Seven-Thirty mine dump (compare to figure 7b). B) Close-up of Seven-Thirty mine dump. C) View of the Seven-Thirty mine dump (at right) looking downstream to the south. Dump is roughly 25 m high. D) Brown Gulch looking downstream to the south from the Griffin monument (see E for location of monument). Horizontal distance from the monument to I-70 is roughly 600 m. E) Brown Gulch and fan looking upstream to the north. F) Fan at the mouth of Brown Gulch. Photo taken April 21, 2003. Compare to Figures 7C and 7D.

A

B

Figure 9. July 28, 1999 debris flows on the south flank of Mount Parnassus. A) Oblique aerial photo of Mount Parnassus. U.S.Geological Survey photograph taken on August 18, 1999 by Intrasearch, Inc. B) Debris-flow deposit on I-70 at the mouth of Watrous Gulch. See vehicles on I-70 for scale. Photo taken on July 29, 1999, by Ed Harp, U.S. Geological Survey.

The flows on the south flank of Mount Parnassus that fed into Watrous Gulch initiated as large rills on steep, non-vegetated slopes above timberline (Figures 10a, b, and c). The matrix of colluvium at the head of the largest rill included 84 percent sand-sized material, 8 percent silt-sized material, and 8 percent clay-sized material (U.S. Geological Survey, 1999, unpublished data). Clasts made up about 50 percent of the colluvium. As the Watrous Gulch debris flow(s) progressed downslope, it eroded material from the channel. Parts of the channel were incised several meters by the flow (Figures 10d, e, f, g). Material eroded from the channel included deposits of layered sandy-silt (Figure 10d), as well as matrix-supported deposits containing sub-angular to sub-rounded boulders (Figure 10d). The matrix of the flow became finer-grained as it progressed downslope. The matrix of the deposit on I-70 (Figure 10h) included 69 percent sand-sized material, 24 percent silt-sized material, and 7 percent clay-sized material. Clasts made up about 70 percent of the deposit on I-70 and were larger and more rounded than those in the colluvium at the source area (compare Figure 10b with Figure 10h).

An analysis of superelevation of debris-flow levees (see Costa (1984) for description of the method) at a bend in the channel of Watrous Gulch directly above I-70 (Figures 10f and 10g) indicated a velocity for the Watrous Gulch debris flow of 32 ft/s (10 m/s). This compares favorably with video footage shot of the debris-flow from I-70 (Denver television station "News 4" footage). This footage shows what appears to be a hyperconcentrated flow coming from the mouth of the basin and mobilizing debris flows on the fan. It is unclear at what time during the debris-flow event the footage was shot.

The debris flow in Watrous Gulch, as well as a debris flow in the gulch directly east of Watrous Gulch, exposed fan stratigraphy that displayed a record of past debris flows (see Figure 13 in Coe et al. (2002)). This stratigraphy indicates that the mean recurrence intervals for debris flows in Watrous Gulch, and the gulch to the east, are about 3,000 yrs and 300 yrs, respectively. The Melton's Numbers for the basins are 0.48 and 1.1, respectively. The negative correspondence between Melton's Number and mean recurrence interval at these two basins fits the overall pattern for the corridor as a whole; that is, fans at the mouths of basins with larger Melton's Numbers have shorter debris-flow recurrence intervals than fans at the mouths of basins with smaller Melton's Numbers (Figure 2). Additionally, the fan stratigraphy at the mouths of the two gulches also suggests that there was at least one previous debris-flow event (between 720 and 930 cal yrs BP) that affected both fans.

SUMMARY

This paper documents historical debris-flow events along the I-70 corridor in Clear Creek County. The majority of the 19 documented events were triggered by rainstorms in the months of July and August. These storms are commonly associated with the flow of moisture from the North American Monsoon. Observations of recent debris flows suggest that the predominant mechanisms of debris-flow initiation in the area are rilling and fire-hose processes.

Figure 10. Photographs of Watrous Gulch taken after the July 28, 1999, debris-flow event. A) Rills in the source area. View to the east. Relief visible is about 610 m (2,000 ft).

Figure 10 – continued. B) Head of the largest rill in the source area (see quart-sized sample bag at lower left for scale). View to the north. C) View upstream along the largest rill. Note geologist for scale. D) Deposits exposed by the July 1999 debris-flow event located about half way between the head of the largest rill and the fan. Channel is about 3 m deep. Matrix-supported boulder-rich deposit on top, sorted, stratified, silt-rich deposit at base. View to the northwest. E) Bedrock-lined channel exposed by the July 1999 debris flow. F) Matrix-supported levee deposits along a bend in the channel above the fan. View downstream to south. Channel depth (thalweg to top of levee) is about 4 m. G) Matrix-supported levee deposits along a bend in the channel above the fan. View upstream to north. H) Fan on I-70. Photos A through D taken on August 4, 1999. Photos E through H taken July 29, 1999.

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Table 1. Historical debris-flow events along I-70 in Clear Creek County. Debris-flow events are listed from east to west and are sequentially numbered. Rainfall data were included as available.

Debris-Flow Event Number	Location	Time and Date of Debris-Flow Event	Trigger	Comments	Source(s) of Information
1	North flank of I-70 at Floyd Hill, just below west-bound lanes	Late July, 2000	Rainfall	Initiated as a soil slip (Campbell, 1975) in highway fill; traveled about 260 ft (80m, slope distance) downslope, and deposited debris just above US Highway 40 (Figure 3).	Personal observations by authors
2	Eastbound lanes of I-70 at Floyd Hill and westbound lanes of I-70 near Silver Plume	Afternoon of July 8, 2001	Rainfall	Debris-flow deposits on I-70 disrupted traffic.	Whaley, 2001
3	Small basins in and near Idaho Springs	Afternoon of June 17, 1993	Rainfall	Debris-flow deposits on westbound lanes of I-70 near Idaho Springs.	Mehle, 1993
4	Small basins in and near Idaho Springs	1:15 p.m. to 2:00 p.m., August 1, 1994	Rainfall, about 1 inch (25 mm) in 45 minutes	Debris-flow deposits on I-70 near Idaho Springs; Virginia Canyon Road closed, probably due to flooding or hyperconcentrated flow.	Garner, 1994
5	Small unnamed basins on southeast flank of Douglas Mountain (Figures 4 and 5)	About 6 pm on August 14, 1983	Rainfall, 1.8 in (45 mm) of rain in 35 minutes at Empire.	Debris covered one westbound lane between mile markers 229 and 232. Both westbound lanes were closed from about 6 to 8 pm.	*Rocky Mountain News*, 1983

Debris-Flow Event Number	Location	Time and Date of Debris-Flow Event	Trigger	Comments	Source(s) of Information
6	Small unnamed basins on southeast flank of Douglas Mountain (Figures 4 and 5)	Night of September 2/3, 1990	Rainfall	Blocked Interstate 70 in both directions east of Georgetown (estimate between mile markers 229 and 232.	*Rocky Mountain News*, 1990
7	Small unnamed basins on southeast flank of Douglas Mountain and east flank of Democrat Mountain (Figures. 4, 5 and 6)	About 2 pm on July 13, 2001	Rainfall, on July 13; mid-day shower (0.21in (5 mm)); afternoon-evening thunderstorms (0.50 in (13 mm)); total rainfall between July 13 (8 am) and July 14 (8 am) was 0.71in (18 mm); total rainfall between July 11 (8 am) and July 13 (8 am) was 0.49 in (12 mm)	Debris closed I-70 for about 4 hrs, debris about 3 m deep on westbound lanes as well as within center median; eastbound lanes closed during clean-up of westbound lanes; highway opened at 6:15 pm.	Multiple, including personal observations by authors; Vaughan and Flynn, 2001; Sherry & Juozapavicius, 2001; rainfall data from Bill Wilson, observer at the National Weather Service station in Georgetown.
8	Unnamed basin(s) on flank of Columbia Mountain	August 3, 1909	Rainfall	Earth washed down Columbia Mountain.	*Georgetown Courier*, 1909
9	Silver Gulch (Figure 4)	About 12 pm, May 28, 1872	Snowmelt	Bank failure dammed Silver Creek near Beecher Silver Mine, dam failed and created a second dam that then failed, triggering a debris flow in Sliver Gulch that flowed to the fan where it damaged several houses.	*Colorado Miner*, 1872

Debris-Flow Event Number	Location	Time and Date of Debris-Flow Event	Trigger	Comments	Source(s) of Information
10	Unnamed basin(s?) on flank of Democrat Mountain	July 28, 1919	Rainfall, 0.7 in (17.8 mm) in 1 hr.	Debris flow closed highway, trapped an automobile, and delayed a train.	*Georgetown Courier*, 1919
11	Small unnamed basins on east flank of Democrat Mountain	Evening of July 27 and/or afternoon of July 28, 1999	Rainfall; total rainfall between July 28 (8 am) and July 29 (8 am) was 0.41 in (10 mm); total rainfall between July 27 (8 am) and July 28 (8 am) was 1.42 in (13 mm) from thunderstorm between 5:40 pm and 10 pm.	Deposits briefly closed westbound lanes of Interstate 70 near Georgetown on afternoon of July 28 (Gutierrez, 1999; Lofholm, 1999); storm system triggered debris flows throughout Colorado (Godt and Savage, 2003; Gutierrez, 1999; Lofholm, 1999).	Multiple, including personal observations by the authors; Gutierrez, 1999; Baca and McCrimmon, 1999; Lofholm and Kirksey, 1999; Lofholm, 1999; News 4 video coverage; Coe et al., 2002; rainfall data from Bill Wilson, observer at the National Weather Service station in Georgetown.
12	Griffith Gulch at Georgetown (Griffith Gulch not shown on maps, but probably located on the west flank of Griffith Mountain (fig. 1) on the east side of Georgetown)	Late May/early June, 1876	Snowmelt?	Debris flow carrying large boulders comes down with "… a roar and force that appalled the inhabitants living in the lower portion of the town."	*Rocky Mountain News,* 1876
13	Unnamed basin(s?) on flank of Republican Mountain	July 26, 1919	Rainfall	Debris flow and rockslide deposited debris (including mine dump debris) about 5 ft (1.5 m) deep near the Kelly Tunnel.	*Georgetown Courier*, 1919
14	Brown Gulch (Figures 7 and 8)	June 13, 1889	Snowmelt	Failure and mobilization of Seven-Thirty Mine dump destroyed ore houses.	*Silver Standard*, 1889

Debris-Flow Event Number	Location	Time and Date of Debris-Flow Event	Trigger	Comments	Source(s) of Information
15	Brown Gulch (Figures 7 and 8)	June 22, 1892	Snowmelt	Failure and mobilization of Seven-Thirty Mine dump; damage to ore houses, blacksmith shops, and Terrible Mill.	*Silver Standard*, 1892
16	Brown Gulch (Figures 7 and 8)	June 26,1895	Snowmelt	Failure and mobilization of Seven-Thirty Mine dump; destroyed Desmoineaux house and Terrible Mill; covered county and mill roads with debris.	*Silver Standard*, 1895
17	Brown Gulch (Figures 7 and 8)	5 am on June 1, 1900	Snowmelt	Debris flow destroyed the shaft house of the Seven Thirty Mine and flowed to Clear Creek. Damage estimated at several thousand dollars.	*Denver Times*, 1900
18	Brown Gulch (Figures 7 and 8)	June 24 and 25, 1912	Snowmelt	Failure and mobilization of Seven Thirty Mine dump; destroyed Griffin cabin, a blacksmith shop, the Lampshire boarding house, the Granite Polishing Works, and the Fox and Hound saloon.	*Georgetown Courier*, 1912; Martin, 1982
19	South and SW flanks of Mt. Parnassus with flows in Watrous Gulch and the gulch immediately to the east (Figures 9 and 10).	Afternoon of July 28, 1999	Rainfall, about 1.4 in (35 mm) in 2 hrs	Rainstorm triggered widespread debris flows in Clear Creek and Summit Counties; flows from Mt Parnassus deposited about 34,000 yd^3 (26,000 $m^{3)}$ of debris on I-70, closing it for about 25 hrs.	Multiple, including personal observations, Gutierrez, 1999; Baca and McCrimmon, 1999; Lofholm and Kirksey, 1999; Lofholm, 1999; NEWS 4 video coverage, Henceroth, 2000; Coe et al., 2002; Godt and Coe, 2003; rainfall data from Grizzly Peak Snotel station near the Arapahoe Basin ski area

COMPUTER SIMULATION FOR MITIGATION OF COLORADO ROCKFALL

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Key Terms: rockfall simulation, computer simulation, rockfall mitigation, rockfall modeling

ABSTRACT

Rockfall is a result of weathering or fracturing on steep natural slopes or rock cuts. Rocks falling from steep slopes usually travel down the slope in one or a combination of free fall, bouncing, and rolling. Rockfall presents a common hazard to transportation routes and structures in Colorado's steep mountainous terrain. Until recently, it was common practice along transportation routes to provide little protection other than posting warning signs. However, as traffic increased in rockfall areas, emphasis on mitigation of the hazard has increased, which has created a need for more understanding of rockfall behavior.

Tools that can accurately predict rockfall behavior are of great value in the design of mitigation schemes. Prior to the development of the Colorado Rockfall Simulation Program (CRSP), selection and design of rockfall protection measures were severely limited, as only Ritchie's ditch design criteria were widely used (although some other alternatives, including computer programs, existed).

Development of CRSP began in the mid 1980s to aid in the prediction of rockfall behavior for the Glenwood Canyon I-70 project. CRSP is a two-dimensional stochastic computer model that is based on field observations and principles of conservation of energy and gravitational acceleration. The program simulates rockfall based on input slope profile, surface roughness, normal coefficient of restitution, tangential coefficient of frictional resistance, rock size, rock shape, and rockfall source zone. These data are collected in the field. The several versions of CRSP have been used successfully as a predictive tool worldwide.

CRSP version 4.0 for Windows was reprogrammed in VISUAL BASIC to minimize early version disadvantages with respect to ease of use. The calibration of program input coefficients, the normal coefficient of restitution (R_n) and the tangential coefficient of frictional resistance (R_t), were also improved in version 4.0. The program re-calibration was accomplished by analyzing rock-rolling data from other investigators and the previous CRSP calibration. This effort resulted in a new set of suggested tangential and normal coefficients.

INTRODUCTION

Rockfall is a common natural geologic process that occurs on steep rocky slopes. As the slope-forming materials weather, individual or multiple rocks are loosened and travel down the slope by one or a combination of free fall, bouncing, and rolling. The individual rocks can vary in size from very small to the size of buses or larger. Autos and structures in the path of rockfall are likely to be severely damaged and personal injuries or fatalities may occur. Until the mid-1980s analysis for mitigation for rockfall along U.S. transportation routes was minimal. Generally, warning signs were posted. Some areas used roadside ditches, designed on the basis of empirical formulas, to catch rockfall. Sometimes, in very high hazard areas, low capacity fences (by today's standards) were installed in an attempt to catch rocks before they reached the roadway or structure. Few tools were available to predict behavior of rocks rolling down an irregular slope that would be helpful for the design of barriers.

For the extension of I-70 through the steep-walled Glenwood Canyon in western Colorado, the Colorado Department of Transportation's (CDOT) baseline geologic studies noted a number of active rockfall areas that threatened the safety of motorists and could badly damage expensive bridge and roadway structures. As a result they wanted to construct appropriate barriers to reduce the risk. The narrow canyon floor didn't allow sufficient room for ditch catchment in most cases. In order to determine the best design and to appropriately locate a barrier or size a catchment ditch, a method was needed to accurately predict rockfall characteristics on irregular slopes. As a result, it was decided to develop a computer program to simulate rockfall.

The purpose of this paper is to present the scientific basis for the Colorado Rockfall Simulation Program (CRSP), a historical summary of over 15 years of program development, and to illustrate how it can be applied to help engineering geologists and engineers design rockfall mitigation.

LITERATURE REVIEW

At the time of the development of version 1.0 of CRSP, a literature review was conducted by Pfeiffer (1989), who found that published literature includes abundant studies dealing with slope stability and rockfall mitigation measures, but papers concerning the mechanics of rockfall motion were limited. Only references that contributed significantly to the development of CRSP will be described in this paper.

In the early 1960s, a rockfall study was conducted by the Washington Department of Highways (Ritchie, 1963). By studying 16-mm films of rockfall, Ritchie observed the importance of angular momentum and bouncing ledges, or "ski jumps" in rockfall. Based on his observations, Ritchie developed criteria for designing cut slopes and ditches that have been widely used (Nichol and Watters, 1983).

Piteau and Associates Limited (1980) wrote and tested a computer rockfall simulation program designed for a main-frame computer. The program used a slope profile divided into straight-line segments, (cells), and laws of motion to determine where a rock will impact the ground. At the point of impact, the velocity of the rock normal to the slope is attenuated by a normal coefficient of restitution, and similarly, motion parallel to the slope is attenuated by a tangential coefficient. The slope of each cell can be adjusted to account for the surface irregularities and angularity of the rock. The program produced velocity and bounce height distributions from the input coefficients, slope geometry, and probability of surface variations.

During the relocation of I-40 in North Carolina, the North Carolina Department of Transportation produced a program to simulate rockfall and to test the effectiveness of widening the roadway ditch to mitigate rockfall hazards (Wu, 1984). Rocks were dropped on an inclined wooden platform and a bedrock slope in order to determine coefficients of "restitution" for motion normal and tangential to the slope. The program randomly varied coefficients to achieve the statistical spread found among rockfall events at a given site. The tests indicated that the rock bounced less with higher impact angles; so, the program reduced the coefficients for larger impact angles.

None of these programs were widely used for predicting rockfall bahavior and design of mitigation. However, it appeared that parts of each program had merit with respect to designing an accurate, easy-to-use rockfall model.

CRSP DEVELOPMENT

CRSP was developed to incorporate the best concepts used by the investigators mentioned above to model the behavior of rockfall. CRSP models the effect of angular momentum noted by Ritchie (1963) by allowing kinetic energy to be transferred between rotational and translational velocity. All of the prior studies noted a statistical variation of rockfall events caused by irregularities of the slope. CRSP approaches these irregularities by using field measurements of surface roughness. The effect of impact angle noted by Wu (1984) is used, which reduces the coefficients according to the velocity normal to the slope. Additionally, CRSP makes adjustments for the difference in friction between rolling and sliding rocks.

Development of the original version of CRSP took place between August 1985 and May 1989. Experimental verification and calibration was conducted by the Colorado School of Mines (CSM) in conjunction with CDOT testing of rockfall fences at a site near Rifle, Colorado. Videotapes recorded the motion of rocks traveling down a slope and impacting the test fence. Research conducted at CSM added graphical data presentations to the program and analyzed the videotapes to verify and calibrate the simulation program.

The original program (version 1.0) was written in BASIC for a DOS operating system by Timothy Pfeiffer for a Master of Engineering thesis in geological engineering at CSM. Version 2.1, 3.0 and 3.0a included minor revisions, corrections and some additions of statistical analyses to the original algorithm (Pfeiffer and Higgins, 1990; Pfeiffer et al., 1991 and 1995). Version 4.0 (Jones et al., 2000) was a complete reprogramming effort using Microsoft VISUAL BASIC® . This program was recalibrated with respect to input coefficients (normal coefficient of restitution and tangential coefficient of frictional resistance) and the new version was compatible with Windows 95 and NT®.

PROGRAM THEORY

Rockfall Parameters

The behavior of rockfall is influenced by slope geometry, slope-material properties, rock geometry, and rock-material properties (Ritchie, 1963). Rockfall events originating from the same source location may behave very differently as a result of the interaction of these factors. Parameters that quantify slope geometry, slope-material properties, rock geometry, and rock-material properties (Table 1) are used to model rockfall behavior.

Table 1. Parameters determining the behavior of rockfall.

Factor	Parameter
Slope Geometry	Slope Inclination Slope Length Surface Roughness Lateral Variability
Slope Material Properties	Slope Coefficients Rock Coefficients
Rock Geometry	Rock Size Rock Shape
Rock Material Properties	Rock Durability

Slope-geometry parameters influencing the behavior of rockfall are slope inclination, slope length, surface roughness, and lateral variability of the slope surface. Slope inclination is critical because it partially defines zones of acceleration and deceleration of the rockfall. Slope length determines the zones in which the rock accelerates or decelerates.

Interaction of a bounding rock with irregularities in the slope surface accounts for most of the variability observed among rockfall events originating from a single source location. These irregularities, referred to as surface roughness, alter the angle at which a rock impacts the slope surface. It is this impact angle that largely determines the character of the bounce (Wu, 1984).

Slope and rock-material properties influence the behavior of a rock that impacts and rebounds from a slope. Numerical representations of these properties are termed the normal coefficient of restitution (R_n) and the tangential coefficient of frictional resistance (R_t), in which the normal direction is perpendicular to the slope surface, and the tangential direction is parallel to the slope surface (Piteau and Associates Limited, 1980; Wu, 1984). When a rock bounces on a slope, kinetic energy is lost due to inelastic components of the collision and friction. While the primary mechanism in resisting motion parallel to the slope is sliding or rolling friction, the elasticity of the slope determines the motion normal to the slope. R_n is a measure of the degree of elasticity in a collision normal to the slope, and R_t is a measure of frictional resistance to movement parallel to the slope.

Because a large rock has greater momentum and is less likely to lodge among irregularities, it will travel farther down a slope than a small rock. Rock size is thus critical in determining the degree surface roughness will affect rockfall behavior. Also important, rock shape contributes to the randomness of rockfall behavior in a manner similar to that of slope surface roughness. Rock shape also influences the apportionment of translational and rotational energy through the moment of inertia.

A critical rock property is durability, which determines whether a rock will break apart upon impact. Rock fragmentation dissipates a large amount of energy and reduces individual rock size. Rock size has a direct relationship to kinetic energy and momentum, which are fundamental considerations in any impact. Two factors act to reduce the influence of rock durability and rock mass on a rockfall. First, the consistency of durability and mass minimizes their effect on the variability of the rock's behavior. Second, the variation of properties among rocks is considerably less than among slopes or even within a given slope.

Program Assumptions

On a natural slope, the parameters discussed above will have a wide range of values and would be cumbersome to analyze as independent variables. CRSP reduces the number of variables by means of the following simplifying assumptions:

1. A slope profile is constructed that follows the most probable rockfall path as established during field investigations (Figure 1). Therefore, all calculations may be in two dimensions.

2. Because the rock type does not change during a rockfall and the range of slope material properties is much greater than that of rock material properties, coefficients assigned to the slope material (R_n and R_t) can account for both the rock and slope properties.

3. The worst-case scenario is generally that of the largest rock that remains intact while traveling down a slope. Therefore, it is assumed that the rock does not break apart in its fall.

4. Rock size and shape are assumed constant for analysis of rockfall from a given source. Values assigned to these parameters are determined by field study of the source area and slope materials.

5. For determination of a rock's volume and inertia, a sphere may be used because it yields a maximum volume for a given radius, which will tend toward a worst case. CRSP will also allow the use of discoidal or cylindrical rocks.

CRSP Algorithm

Rockfall simulation begins within a selected vertical zone on the slope profile (Figure 1) that represents the source location by assigning to a rock nominal initial horizontal and vertical velocity components. The velocity components are acted upon by gravitational acceleration until the rock's trajectory intersects the slope below at resultant velocity V_1 (Figure 2). At each impact, the incoming velocity, impact angle, and rotational velocity are used to calculate new velocity components and rate of rotation. At the point of impact, the slope angle (ϕ) is randomly varied up to the limit set by the maximum probable variation in the slope (θ_{max}). This limit is determined by field observation of the slope surface. The surface roughness (S) is defined as the

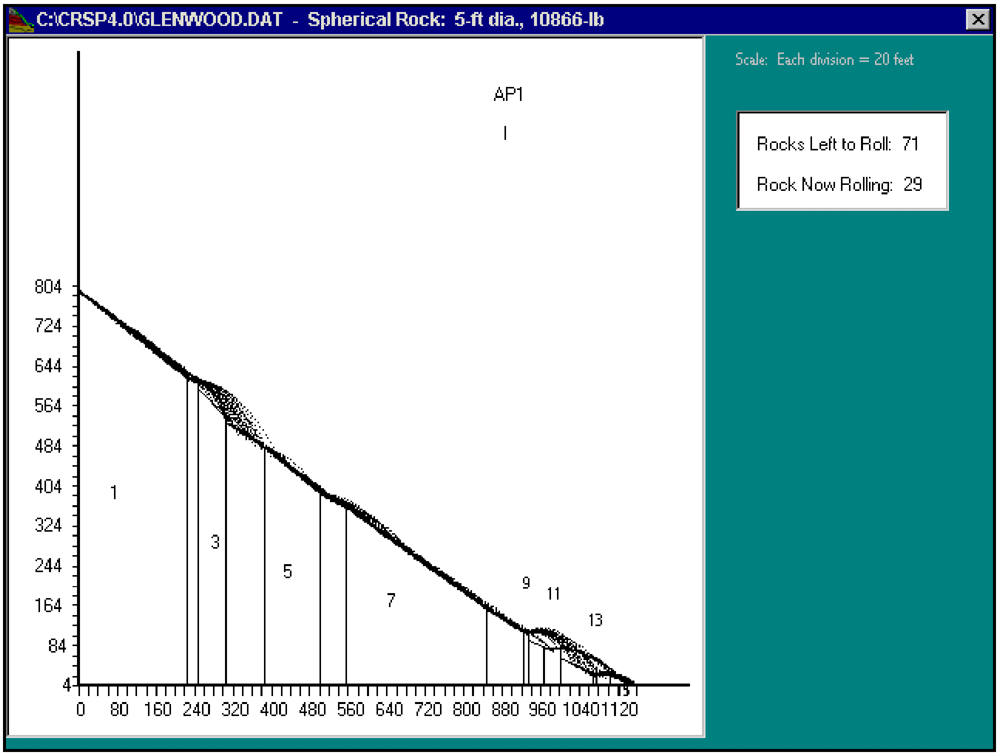

Figure 1. CRSP slope profile window (with rockfall simulation in-progress).

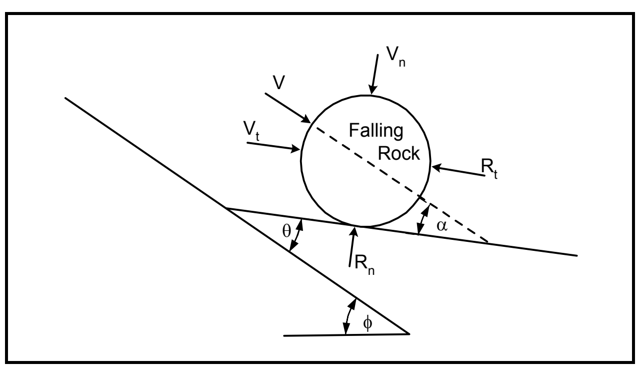

Figure 2. Impact angle (α) defined as a function of rock trajectory, slope angle (ϕ), and slope variation (θ). Rock velocity (V) is reduced into normal (V_n) and tangential (V_t) components. The tangential coefficient of frictional resistance (R_t) and the normal coefficient of restitution (R_n) act to decrease the falling rock's velocity (Pfeiffer, 1989; Pfeiffer et al., 1991; 1995).

perpendicular variation of the slope within a slope distance equal to the radius of the rock (Figure 3). This describes the slope angle experienced by the rock on impact. Surface roughness (S) and rock radius (R) are used in calculating the maximum-allowable variation in slope angle (θ_{max}) by equation 1.

$$\theta_{max} = \tan^{-1}\left(\frac{S}{R}\right) \tag{1}$$

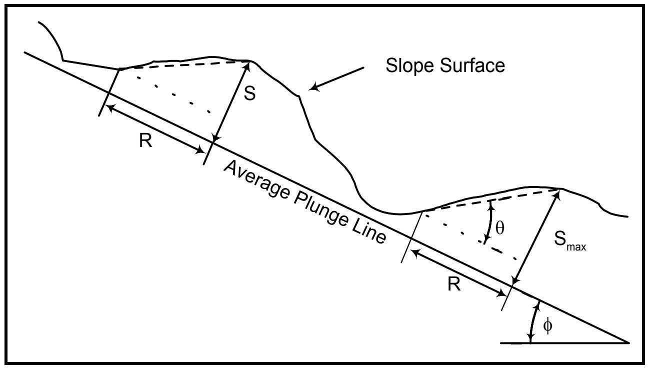

Figure 3. Surface roughness (S) established as the perpendicular variation from an average plunge line (defined by slope angle ϕ) over a distance equal to the radius of the rock (R). Maximum slope variation (θ_{max}) is defined by S and R (Pfeiffer, 1989; Pfeiffer et al., 1991; 1995).

The angle of variation (θ) is a randomly selected angle, less than θ_{max}, that determines the variation in the slope angle (ϕ). This random variation is largely responsible for the statistical variation of rockfall events modeled. The impact angle (α), is used to resolve the incoming velocity (V_1) into velocity components tangential ($V_{t1} = V_1\cos\alpha$) and normal ($V_{n1} = V_1\sin\alpha$) to the slope surface (Figure 2).

A new tangential velocity is calculated from the conservation of energy considerations in equation 2.

$$\left(\frac{1}{2}I\omega_1^2 + \frac{1}{2}MV_{t_1}^2\right)f(F)SF = \frac{1}{2}I\omega_2^2 + \frac{1}{2}MV_{t_2}^2 \tag{2}$$

where:

M	=	rock mass
I	=	rock moment of inertia
I	=	$\frac{2MR^2}{5}$ (for a sphere)
I	=	$\frac{MR^2}{2}$ (for a disk)
I	=	$\frac{MR^2}{4} + \frac{ML^2}{12}$ (for a cylinder, L = length)
ω_1	=	initial rotational velocity
ω_2	=	final rotational velocity
V_{t1}	=	initial tangential velocity

V_{t2}	=	final tangential velocity
f(F)	=	Friction function
	=	$R_t + \dfrac{1 - R_t}{\left(\dfrac{V_{t_1} - \omega_1 R}{20}\right)^2 + 1.2}$
SF	=	Scaling factor
	=	$\dfrac{R_t}{\left(\dfrac{V_{n_1}}{250 R_n}\right)^2 + 1}$

In any non-perfectly elastic collision, kinetic energy is lost. In the case of a rock impacting a slope, the component of kinetic energy parallel to the slope and the rotational energy are attenuated by friction along the slope and collisions with features perpendicular to the slope. Friction is a function of the slope material, determined by the tangential coefficient and whether the rock is initially rolling over or sliding upon the surface. The friction function adjusts the tangential coefficient according to the velocity at the surface of the rock relative to the ground at the beginning of the impact. Figure 4 shows a graph of the friction function.

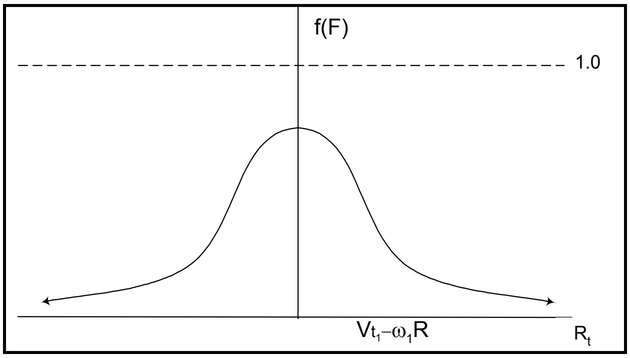

Figure 4. Friction function $f(R_t, V_{t1} - \omega_1 R)$ as a function of the difference between tangential and rotational velocities (Pfeiffer, 1989; Pfeiffer et al., 1991; 1995).

Another major influence on the loss of kinetic energy tangential to the slope is the velocity normal to the slope. An increase in velocity normal to the surface results in a greater normal force during impact. The scaling factor adjusts for the increased frictional resistances due to an increase in the normal force.

Equation 2 may be solved for the new tangential and rotational velocities by establishing the relationship between rotational velocity and tangential velocity shown by equation 3.

$$V_{t_2} = \omega_2 R \tag{3}$$

Equation 3 describes the situation where the rock rolls across the surface during impact rather than sliding. Observations of bouncing rocks show that regardless of the initial rotational velocity, rocks always leave the surface in the rolling mode. The relationship in equation 3 allows rotational energy to be applied to tangential velocity, or tangential velocity to be applied to rotational velocity. The energy lost during the bounce is determined from the difference between rotational and tangential velocities, the velocity normal to the slope, and the tangential coefficient. Constants used in the friction function and the scaling factor were determined by experiment. Solving equation 2 for the new tangential velocity yields equation 4.

$$V_{t_2} = \sqrt{\frac{R^2\left(I\omega_1^2 + MV_{t_1}^2\right)f(F)SF}{\left(I + MR^2\right)}} \tag{4}$$

A new normal velocity (V_{n2}) is established by equation 5.

$$V_{n_2} = \frac{V_{n_1} R_n}{1 + \left(\frac{V_{n_1}}{30}\right)^2} \tag{5}$$

This equation uses the coefficient of restitution (R_n) and a velocity-dependent scaling factor ($1/(1 + V_{n1}/30)^2$) to determine the new normal velocity (V_{n2}).

The normal scaling factor (B), graphically represented in Figure 5, adjusts for the decrease in normal coefficient of restitution as the impact velocity increases. This factor represents a transition from more elastic-rebound at low velocities to much-less-elastic rebound caused by increased fracturing of the rock and cratering of the slope surface at higher impact velocities (Habib, 1976).

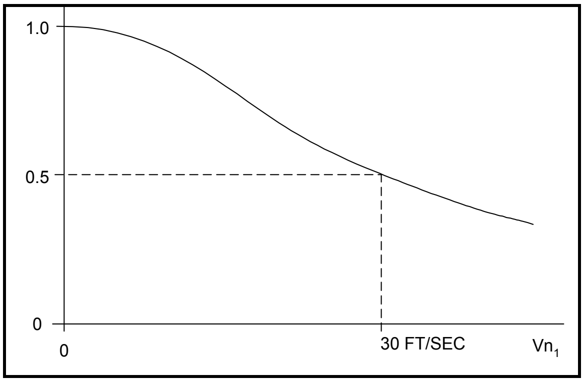

Figure 5. Normal coefficient scaling factor, $B = \frac{1}{1 + \left(\frac{V_{n_1}}{30}\right)^2}$, as a function of the incoming normal

After each bounce, CRSP performs an iteration to find the time elapsed until the next bounce. Elapsed time is calculated from x- and y-velocity components, gravitational acceleration, and the slope profile. After a new impact position is established, the next bounce is calculated as before. If the distance the rock travels between bounces is less than its radius, it is considered to be rolling and is given a new (x, y) position equal to a distance of one radius from its previous position. This models a rolling rock as a series of short bounces, much like an irregular rock rolls on an irregular surface.

Sensitivity to Input Parameters

With multiple parameters affecting the simulation results, it is difficult to understand the effects of each on the results. Computer modeling usually includes several simulations, using a range of possible input parameters. It is often helpful when choosing a range of input parameters to know what effect each has on the results. By varying only one input parameter at a time for a site of interest, the effect of each parameter may be observed.
velocity (Pfeiffer, 1989; Pfeiffer et al., 1991; 1995).

On a uniform slope, rock size will not affect rockfall behavior. Natural slopes are not usually uniform, and thus the size of the rock does have an affect. On portions of the slope where the rock's velocity is decreasing, a large rock having more momentum will require more distance to slow down than a relatively small rock. Another reason large rocks travel farther and faster than small rocks is the effect of surface roughness. While the surface roughness is proportional to the rock size, on most slopes the surface roughness will increase the impact angle more for small rocks than for large rocks. The larger the impact angle, the more energy the rock will lose during impact. By itself, rock size does not affect the results, but it does affect the influence of changes in slope angle and surface roughness.

Simulation results from the area used for the original experimental testing of CRSP show a gradual increase in average velocity with increasing rock size (Pfeiffer, 1989). Typically, simulation results on modeled slopes show many of the rocks stopping on the slope when a smaller rock size is used, whereas results for larger rocks show the rocks traveling the length of the slope. This variation is consistent with observations of the stopping position of rocks on these slopes.

As expected, slope angle is the most important factor in determining the behavior of rockfall. Falling rocks will tend to increase in velocity up to a maximum, depending on slope angle. A general pattern of increase followed by a leveling off is observed for both velocity and bounce height (Pfeiffer, 1989; Jones et al., 2000).

The effect of surface roughness changes with slope angle. An increase in surface roughness will have a greater effect on low angle slopes than on steep slopes. An increase in surface roughness will also generally result in a decrease in velocity and an increase in bounce height until the surface roughness decreases the velocity to the point where bounce height also begins to decrease. The surface roughness value where bounce height begins to decrease is lower for smaller slope angles.

Material coefficients affect rockfall behavior by determining the amount of energy absorbed during impact, with high coefficient values resulting in lower energy loss during impact. Because the coefficients only act on impact, their effect on bounce height and velocity is dependent on the number of bounces. On steep slopes, where rocks impact the slope less often,

the effect of the coefficients on rockfall behavior becomes negligible. The effect of the coefficients on rockfall behavior is largest for low-angle slopes, where the rockfall velocity is decreasing. On most slopes, changes in the coefficients, within reasonable limits for a specific slope material, will not produce a significant change in results.
Several factors act to reduce the effect of surface material properties on rockfall behavior. First, the effect of slope angle and surface roughness is so much greater than the effect of material properties that the results of changes in coefficients can be obscured. Second, the coefficients are modified by factors, discussed above in the "CRSP Algorithm" section, that also tend to obscure the results of changes in the coefficients. The most important factor that modifies the coefficients is the velocity normal to the slope at impact. The normal velocity is dependent on the impact angle, which is determined by the slope angle and surface roughness. For these reasons, the effect of changes in coefficients is largely dependent on the slope configuration. Therefore, the recommended method of determining the sensitivity to changes in coefficients is to test the effect of changes in coefficients at the specific site of interest by varying the input parameters within a range consistent with properties reasonably attributable to the site.

VERIFICATION AND CALIBRATION

During development of CRSP (Version 1.0), field tests were conducted for program verification and calibration. A test site near Rifle, Colorado, was used to collect rockfall data. Rocks were rolled down a 300-ft-high (91-m) hillside consisting of thin desert soil with rocky ledges. The very sparse vegetation had little effect on rockfall behavior. A worst-case slope profile was used to compare CRSP-predicted rock velocities and bounce heights with field data. The actual rock-rolling data were compared to CRSP output, and it was found that CRSP-predicted maximum bounce height closely matched field observations while the CRSP-predicted maximum velocity was substantially low. Thus, the Rifle experimental data were used to adjust the constants in the friction function and scaling factors until the simulation data fit the experimental values for travel time, number of bounces, and bounce height.

Also, CRSP simulations were compared to field trials conducted by the California Department of Transportation (McCauley et al., 1985) and to Ritchie's (1963) rockfall-ditch design criteria. It was found that CRSP version 1.0 predictions tended toward more of a worst case scenario than do the studied field tests, but the overall conclusions were similar.

Another literature review was conducted at the time of the development of version 4.0. Findings included a few papers that analyzed CRSP's accuracy and sensitivity. Larsen (1993) conducted a study of various rockfall models. As part of his study, he performed sensitivity analyses of CRSP's predicted rockfall runout distance, bounce height, and velocity. Surface roughness was found to be extremely sensitive, while normal coefficient of restitution and rock size were also established to be important. Conversely, rock shape, initial velocity, and tangential coefficient of frictional resistance were found to have only minor influences on the output. Overall, Larsen found CRSP to have practical applications to two-dimensional rockfall investigations.

Evans (1989) compared several rockfall models as part of a study to design catch bench geometry for open-pit mines. He found that CRSP was consistent in predicting rockfall behavior on eight different test slopes. Evans thus recommended CRSP for use in designing catch bench geometry in surface mines. Evans also performed a sensitivity analysis of the program input parameters and found that the tangential coefficient of frictional resistance was not especially sensitive while the normal coefficient of restitution was somewhat sensitive. However, Evans asserted that surface roughness was the most sensitive input parameter.

The Oregon Department of Transportation (Pierson et al., 1994) used CRSP to aid in the planning of research for rockfall protective ditch design for 0.25 (horizontal): 1 (vertical) slopes. Field test results were compared with CRSP output to evaluate whether the model was reasonable for the given application. Once CRSP's reliability was established, the program provided a means to extrapolate velocity and bounce-height information for slopes other than the 40- to 80-ft-high (12- to 24-m) slopes tested and modeled.

During development of version 4.0, additional program calibration was conducted using rock-rolling data from departments of transportation (primarily California and Colorado) and from Swiss rockfall researchers. The data were chosen based on availability, slope-forming materials, and slope angle. An attempt was made to provide calibration information for varieties of slope properties not well represented by existing data.

CONSTRUCTING A ROCKFALL SIMULATION

CRSP version 4.0 (Jones, et al., 2000) yields estimates of probable rockfall velocities, bounce heights and kinetic energies. After over 15 yrs of worldwide use, the program has become a standard tool used for designing rockfall mitigation. Various combinations of cut slope, barrier, and ditch configurations and locations can be tested until a configuration is found that is both aesthetically acceptable and safe with respect to rockfall. Calculation of energies developed by rockfall are useful in designing barriers, and in combination with bounce-height predictions, is useful in determining optimal locations for barriers.

CRSP requires the following input data:

1. A slope profile, input as a series of straight-line segments, referred to as cells, designated by the Cartesian (x, y) coordinates of the endpoints of each line (Figure 1).

2. An estimation of the roughness of the slope surface (relative to rock radius) within each cell (Figure 3).

3. Coefficients (Rt and Rn) that determine the rock energy loss upon slope impact.

4. The size, shape, and starting location of the rocks comprising the rockfall events.

CRSP uses these input data in a stochastic model to produce statistics on probable rockfall velocity, kinetic energy, and bounce height based on a series of rock rolls under identical conditions. The following data are output by CRSP:

1. A slope profile that shows cell locations and the position of each simulated rock every tenth of a second as it travels down slope.

2. The maximum, average, minimum, and standard deviation of rock velocities at each of one to three selected points (analysis points) on the slope.

3. The maximum, average, and standard deviation of rock velocities at the end of each cell.

4. The maximum, average, geometric mean, and standard deviation of rock bounce-heights at each analysis point.

5. The maximum and average bounce heights at the end of each cell.

6. The maximum, average, and standard deviation of kinetic energies at each analysis point.

7. Cumulative probability analyses of velocity, kinetic energy, and bounce height at each analysis point.

8. The number of stopped rocks in each 10-ft or 10-meter slope interval.

9. Various graphs illustrating the output.

Field Data Collection

Rockfall hazard areas can be identified by examining slopes for evidence of recent rockfall events. A comprehensive investigation should examine the slopes for potential source zones, such as highly fractured or weathered rock masses or zones of accumulation such as talus slopes.

Selection of input parameters begins with identification of the rockfall path from the source area to the area that may require protection. If more than one potential rockfall path is present, then multiple slope profiles may be required. The profile of this path is input into CRSP as a series of straight-line segments called cells. This profile may be obtained from surveying the slope or from detailed large-scale topographic maps. Division of the profile into cells and refining the profile is best done in the field, where changes in slope and slope-material properties can be observed. Values for surface roughness, tangential coefficient, and normal coefficient must be selected for each cell. These are initially estimated from tables of values that have been derived empirically from field-calibration experiments (Jones et al., 2000). The initial values are refined through model calibration analyses for the specific site being modeled. Also, cell boundaries and rock sizes must be chosen.

Rock-Size Determination- The size of the rocks involved in rockfall events depends on the size of the blocks in the source area and on the durability of the rocks. While it is conceivable that a rock breaks during descent or a smaller rock could produce a worst case, the worst case is usually for the largest rock that travels the length of the rockfall path. The largest rocks found at the base of the rockfall path that can be identified as having fallen from the source area should be the selected size. Also, rock size can be determined from the source area by measuring joint spacings. The rock type or types should be noted and will aid in the choice of appropriate rock density.

Cell-Boundary Selection- Cell boundaries are used to define the slope profile and areas of uniform slope and characteristics. Cell boundaries are selected where changes in slope occur and/or where the slope-material properties change. The number of cells to use depends on the length and complexity of the slope. Too few cells will decrease the accuracy of the simulation, but too many cells make the investigation needlessly difficult. Closely spaced cells may be inappropriate, because smaller variations in the slope are modeled by the surface roughness. Also, cell configurations that require excessive precision may result in erroneous outputs because the variables in the program are single precision. The influence of changes in slope becomes smaller with distance; therefore, more detail is put into the slope profile near the area where mitigation is being considered.

Surface Roughness- Surface roughness is a function of the size of the rock and the irregularity of the surface. Surface roughness is an estimation of how much the slope angle may vary within the radius of the rock (Figure 3). The beginning rockfall investigator may want to take some measurements of surface roughness. This may be done by stretching a measuring tape along the slope surface (within each cell) and measuring the distance to the slope perpendicular to the tape. Within each slope distance of one-rock radius, the greatest measurement that occurs with some frequency is the surface roughness. With a little practice, an estimation of the surface roughness may substitute for these time-consuming measurements.

Because the program selects an impact angle variation up to the value defined by the surface roughness, the largest probable surface roughness should be used. This is not always the value for the largest bump on the slope, or an average variation in the slope; rather, it is the value of the largest variation that occurs with some frequency. A range of probable surface roughness values should be selected for each cell, and if more than one rock size is being considered, separate surface-roughness values are collected for each rock size. On very smooth surfaces, such as pavement, surface roughness is a function of the irregularity of the rock. In such cases, appropriate surface roughnesses will typically be between 25 and 50 percent of the rock radius. One case for which surface roughness is extremely important is talus slopes. In all cases, a range of probable surface roughness values should be collected for use in a sensitivity analysis.

Tangential Coefficient- The tangential coefficient of frictional resistance determines how much the component of the rock's velocity parallel to the slope is slowed during impact. Vegetation and, to a lesser extent, slope material influence the tangential coefficient. A range of probable values should be selected for each cell, for use in a sensitivity analysis of the slope. Jones et al. (2000) suggested ranges of tangential coefficient (R_t) values for various slope materials that are based on field testing. The tangential coefficient is significantly less sensitive than the normal coefficient, but the tangential coefficient may become more important for vegetated slopes. Tangential coefficient values for slopes with vegetation more than a few feet tall are difficult to assess. The coefficient for an individual rock may be low; however, the first rocks down the slope clear a path for the next rocks.

Normal Coefficient- The normal coefficient of restitution is a measure of the change in the velocity normal to the slope after impact, compared to the normal velocity before the impact. The normal coefficient is determined by the rigidity of the slope surface. Jones et al. (2000) suggested ranges of normal coefficient values for different materials. They found that the normal coefficient appears to be somewhat dependent on slope length, with a longer slope corresponding to a greater value of R_n. The normal coefficient is particularly sensitive compared to the tangential coefficient.

Site-Specific Calibration- In order to achieve the highest degree of accuracy from CRSP, the program should be calibrated to each specific study site. This can be accomplished by first estimating probable ranges of surface roughness, tangential coefficients, and normal coefficients for the slope. The ranges can then be input along with the rest of the collected data, and the output compared to field observations. For example, if rocks are recognized to frequently stop at particular locations on the slope, CRSP should be in accord. Similarly, if rocks, which have fallen from the slope, are observed 25 ft (7.6 m) from the slope base, CRSP should not show that all rocks of that size stop on the slope. The user can adjust surface roughness, tangential coefficient, and normal coefficient combinations to model what they see in the field. Because surface roughness can be directly measured or estimated in the field and the tangential coefficient is generally not very sensitive, the calibration should concentrate primarily on the normal coefficient. However, as discussed earlier, the tangential coefficient may be sensitive for significantly vegetated slopes.

Field Testing- Large projects, such as corridor improvements, may justify full-scale testing of actual rockfall behavior, which is used for program calibration. The analysis of actual rock rolling is often done in conjunction with the removal of loose material as part of an overall rockfall mitigation program. Although time consuming and costly, these efforts can be very useful in determining the appropriate range of values for the normal and tangential coefficients.

The field-testing program involves marking the slope with a series of reference lines at regular intervals that are perpendicular to the slope's plunge. Video cameras are installed to capture the time duration from each reference line and ultimately to determine the velocity of the rocks. If high-speed cameras are used, rotational velocity can also be captured.

Before the rocks are rolled, individual rocks are measured along three axes to determine the dimensions of the rocks. The weight of each rock can be calculated using its estimated specific gravity or more accurately measured with a load cell, if desired.

The video captures the initial velocity the moment the rock is pushed from the source area. It also allows the bounce height, bounce length, translational velocity and rotational velocity to be determined as the rock travels down the slope. From this, the total kinetic energy of the rockfall can be calculated. This information can then be compared with the analysis performed by CRSP and the input parameters can be adjusted to fit the actual site conditions.

Once the model is calibrated to the site, slope modeling can be performed and mitigation measures can be designed using the appropriate values for kinetic energy and bounce height.

APPLICATIONS OF CRSP

Glenwood Canyon, Colorado

During the construction of the I-70 Corridor Project through Glenwood Canyon, Colorado, natural rockfall events were monitored. A database was established for the areas that received the highest incidence of rockfall. These data, and extensive field review, formed the basis for prioritization of the rockfall-prone areas. CRSP was used to analyze these areas and to assist in determining the most suitable types and locations of mitigation systems.

In general, three protection systems were utilized in Glenwood Canyon: attenuators, fences, and barriers. Prior to construction, these systems underwent extensive field-testing and were proven to be effective in controlling rockfall. A total of 31 protection systems were installed along the 12-mile (19-km) corridor. Of this total, 28 fences, three attenuators and three barriers were used. Based on energy and bounce-height data obtained from CRSP, energy capacity and effective height were selected for the systems.

The rockfall-attenuator system was designed to absorb kinetic energy and reduce bounding heights from incoming rockfall and then to return to its original position without maintenance intervention. The system utilized columns of used tires on rims supported on a series of steel pipes. The pipes are attached to a large-diameter wire rope that is suspended across a gully or draw. Rock anchors were used to secure the wire rope assembly to bedrock on either side. A facade consisting of wooden posts suspended from a separate wire rope was placed down slope of the tire elements to address aesthetic concerns. Locations that are best suited for the attenuator are in rockfall chutes located near the upper extent of a talus deposit. Rocks detach from the source area and encounter the tire attenuator while energy in the rock is at or near maximum. After impact, most of the energy is absorbed, thus increasing the probability that the rock will be deposited on the talus and will not travel down to the roadway.

CRSP integrates with this system by providing the optimum location based on the incoming velocity and bounding height. Analyzing the kinetic energy of the rock at the designated location and subtracting the energy lost to the system may also determine the effects of the attenuator on the resulting rockfall. The rock is then restarted from this location with the reduced velocity and modeled for the effects that the remaining slope will have on rockfall behavior. Thus the location of the attenuator was identified by repeating the program until the desired level of mitigation was achieved.

Several fence systems were installed at various locations throughout the canyon. They included a high-capacity ring net, a medium-capacity cable net, and a low-capacity wire-mesh system. Each of the fences installed were flexible systems that absorb the energy from the rock in the cable or mesh panels. The energy is then transferred from the panel through a series of cables anchored to the ground.

To evaluate potential rockfall fence locations, CRSP was used to determine the locations along the chute that had the lowest modeled energy. The locations were examined to determine the constructability and visual impacts. The final locations of the fences were placed in areas not easily seen from I-70, but still providing the ability to restrain the majority of rockfall passing through the chute.   A CRSP analysis was conducted at each of the final fence locations and the type of fence system was chosen based on the energy modeled at that specific point. Figure 6 represents a typical output from CRSP. The results of the selected analysis point (AP1) are shown in Table 2.

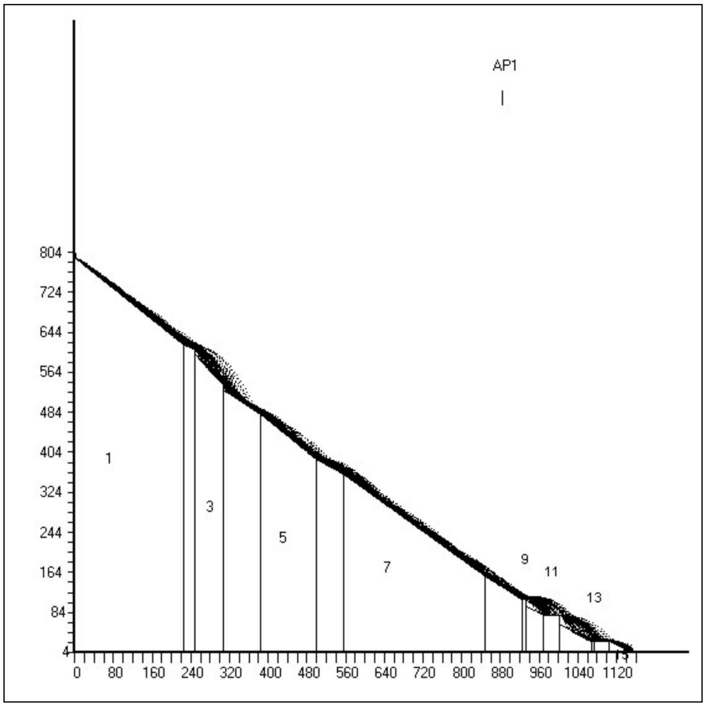

Figure 6. Typical CRSP cross section for Glenwood Canyon.

Table 2. Analysis point 1 (AP1) data.

Velocity (ft/sec)	**Bounce Height (ft)**	**Kinetic Energy (ft-lb)**
Maximum: 92.13	Maximum: 15.56	Maximum: 379003
Average: 71.24	Average: 5.75	Average: 242420
Minimum: 52.32	G. Mean: 3.52	Std. Dev.: 60226
Std. Dev.: 10.08	Std. Dev.: 4.87	
Analysis Point 1: X = 885, Y = 134		

Rockfall barriers constructed in Glenwood Canyon were designed using similar requirements of mechanically stabilized earth (MSE) walls. The barriers were constructed using concrete modular blocks or wooden timbers as forming elements. The forming elements, which also served as the facing, were tied together and reinforced with geosynthetics and backfilled select material. Configurations of the barriers ranged from 6-ft (1.8-m) wide by 6-ft (1.8-m) high to 8-ft (2.5-m) wide by 10-ft (3-m) high. By using reinforced soil as backfill, the barriers are semi-flexible and allow the damping of high-energy rockfall events.

Locations for the MSE barrier systems were selected based on constructability and expected rockfall energies as predicted by CRSP analyses. At the selected sites, the program indicated that 10- to 20- ft-wide (3- to 6-m) ditches would be required in front of the impact walls, with the impact surface constructed to near vertical. The wall height and vertical configuration were necessary to insure that the large boulders would not climb the wall face due to the high rotational velocity. Behind the wall the slope was configured to direct future rockfall events toward the wall and to blend the feature into the hillside.

CRSP was instrumental in analyzing complex slope conditions in and around the Glenwood Canyon Project and in determining the most-appropriate and cost-effective methods for controlling hazardous rockfall. Through the installation of these mitigation systems, rockfall-related traffic incidences have dropped substantially and observed rockfall frequency on the roadway has diminished.

Georgetown, Colorado

The Georgetown Incline is located along I-70 between the towns of Georgetown and Silver Plume, Colorado. A study was conducted to evaluate the potential sources for rockfall events, analyze the rockfall hazards, and present mitigation options for the highway.

The adjacent slope along the incline is very steep with numerous outcrops and rock cuts. The rock cuts extend along almost the entire segment with heights greater than 100 ft (30 m). Approximately 100 rockfall related accidents have occurred in the past 24 yrs with 17 injuries and three fatalities along the incline. There is more rockfall activity than reported accidents, which is evident from the filled ditches along westbound I-70 and the boulders and cobbles covering the embankments of eastbound I-70 (opposite the rock slope).

Geological hazards were assessed along this section of the I-70 corridor. The work included field reconnaissance and collection of climatologic and accident data. Outcrops, rockfall chutes, and rock cuts were mapped and evaluated for potential rockfall. Many areas were identified as having high rockfall potential and were evaluated using CDOT's Colorado Rockfall Hazard Rating System (CRHRS), a modified Q-rating system, risk analysis, field observations, aerial photos, and geologic maps.

Once the rockfall source zones and chutes were identified, CRSP was used to evaluate rockfall behavior and assist in determining feasible mitigation methods. The profiles were determined from topographic maps, observations, and survey data. Most chutes are surfaced by vegetation at the top and bedrock toward the bottom. All analysis points were placed 20 ft (6 m) above the steep rock cuts (directly above the road) and a design rock of 3-ft-diameter (1-m) was chosen. The 3-ft-diameter (1-m) rock size was based on site observations and was used to compare all the mapped chutes. The analysis points were chosen at the most feasible location for rockfall fences. A fence located closer to the rockfall source would be more efficient, but would be virtually impossible to maintain and difficult to construct.

Figure 7 shows a typical cross section evaluated by CRSP modeling on the Georgetown Incline. Tables 3 to 5 represent the data obtained at three analysis points evaluated. Analysis point 1

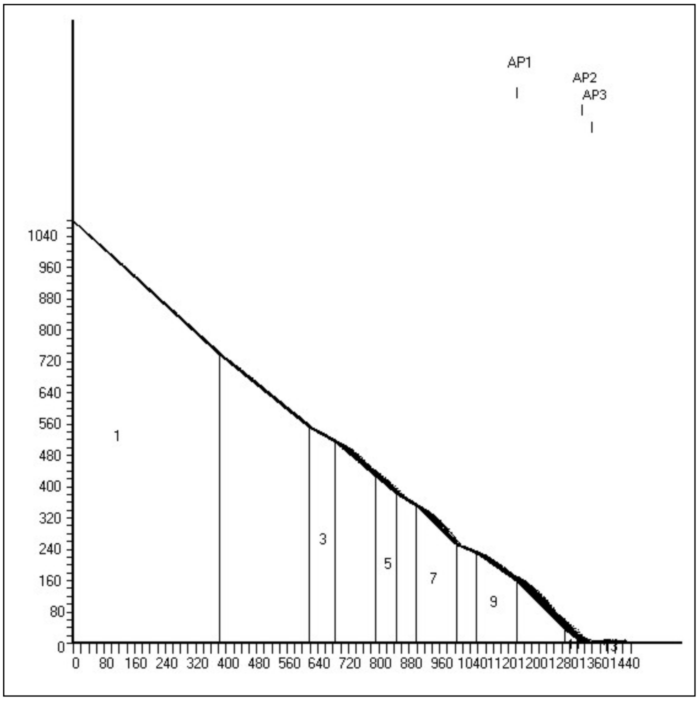

Figure 7. Typical CRSP cross section of the Georgetown Incline project.

Table 3. CRSP AP1 data.

Velocity (ft/sec)	Bounce Height (ft)	Kinetic Energy (ft-lb)
Maximum: 83.94	Maximum: 12.06	Maximum: 301350
Average: 63.19	Average: 5.33	Average: 187044
Minimum: 48.94	G. Mean: 3.43	Std. Dev.: 36736
Std. Dev.: 7.16	Std. Dev.: 3.34	
Analysis Point 1: X = 1161.6, Y = 158		

Table 4. CRSP AP2 data.

Velocity (ft/sec)	Bounce Height (ft)	Kinetic Energy (ft-lb)
Maximum: 111.56	Maximum: 22.73	Maximum: 525622
Average: 74.11	Average: 6.94	Average: 260570
Minimum: 42.38	G. Mean: 5.09	Std. Dev.: 79490
Std. Dev.: 12.85	Std. Dev.: 2.48	
Analysis Point 2: X = 1334, Y = 0		

Table 5. CRSP AP3 data.

Velocity (ft/sec)	Bounce Height (ft)	Kinetic Energy (ft-lb)
Maximum: 74.33	Maximum: 4.49	Maximum: 264576
Average: 27.08	Average: .41	Average: 45762
Minimum: 6.32	G. Mean: .11	Std. Dev.: 45524
Std. Dev.: 12.85	Std. Dev.: 8.73	
Analysis Point 3: X = 1358, Y = 3		

(AP1) was chosen for the fence location based on moderate-energy values and lower bounce heights as compared to AP2. AP3 was chosen to model rocks that reach I-70.

During 1999, a year of high precipitation, two large rockfall events occurred, one in May and the second in December, both resulted in fatalities. It was reported that during the failures, boulders the size of bathtubs fell on the highway and bounced beyond to the adjacent bike path. The locations of these events were modeled using CRSP, which indicated that if material were to reach eastbound I-70 or the bike path, the rocks must have had relatively high velocities and bounce heights.

Following the evaluation, three fence systems were located near the areas of high rates of rockfall accidents. The systems installed were fences constructed with either cable net or ring nets (Figure 8). A 3-foot-high (1-m) gap was designed at the bottom of the fence to aid in clean out of the system. Draped cable net was attached above the gap to contain the fallen material and direct it to the roadside ditches. Although these systems will not completely eliminate the risk of rockfall at the selected locations, they will help minimize the potential of rocks reaching the highway.

Figure 8. Rockfall fence located on the basis of CRSP analysis.

CONCLUSIONS

CRSP allows modeling of rockfall on any shape of slope. The program is a stochastic model that produces statistics on probable rockfall velocity, kinetic energy, and bounce height based on a series of rock rolls under identical conditions. The program considers slope irregularities, slope surficial material properties, rock size and shape, and both translational and rotational energy of the bounding rock. Program-input values are easily obtained from simple field observations, which allows rapid, inexpensive, site-specific analyses of rockfall behavior. The program has proven to be helpful for determining the optimum rockfall catchment-ditch dimensions and locations, rockfall-barrier size and location, and barrier capacities. None of these predictions were easily obtained prior to the development of the program. CRSP has been in use worldwide for over 15 yrs on a variety of projects. Much of the experience gained in that time has been used to improve the various program versions.

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RED ROCKS AMPHITHEATRE: 2001-2003 RENOVATION RETAINING WALLS

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Key Terms: Mechanically Stabilized Earth (MSE) wall, soil nail wall, ground anchor (tieback) wall, Fountain Formation, reinforced soil slope (RSS), compaction grouting, retaining walls

ABSTRACT

The City and County of Denver renovated Red Rocks Amphitheater to add a new Visitor Center and update its infrastructure, telecommunications, and accessibility. Yeh and Associates provided retaining wall design and slope stability mitigation for the project.

Highly variable subsurface conditions and limited access compounded the design process. Materials onsite range from very competent sandstone to loose silty sand with voids. During Red Rock's original construction during the late 1930's, fill material was loosely placed.

The Visitor Center was designed with soil nail walls ranging in height from 10 to 35 feet in cut sections adjacent to the amphitheater seating. A mechanically stabilized earth (MSE) wall was designed in the western fill section. The MSE wall ranges from two feet to 35 feet in height, varying with the natural topography.

The South Stairs and Concession Area was designed with three separate ground anchor (tiebacks) walls to mitigate the potential movement of a steep slope and adjacent structures. Compaction grouting and reinforced soil slopes (RSS) were used to mitigate settlement of this area.

INTRODUCTION

Red Rocks Amphitheater underwent renovations in 2001 through 2003 to improve the facilities and to repair damage that has occurred over the years. Construction focused on updating the water and sewer infrastructure, upgrading available power levels and telecommunications capabilities, improving the venues accessibility for those with disabilities, and constructing a new visitor center. There appeared to be some movement in the seating area on the south side of the amphitheater, possibly due to slope instability or settlement of fill materials. This movement impacted an historic slope south of the seating area, which needed to be preserved. Yeh and Associates was contracted by the project architect to design the cut and fill retaining walls associated with the renovations as well as mitigate the movement of the south slope. The following sections detail the design and construction challenges that were encountered in the development of these retaining walls.

LOCATION

Red Rocks Amphitheatre is located in Red Rocks Park near the town of Morrison, approximately 15 mi west of Denver along the Front Range of the Rocky Mountains (Figure 1).

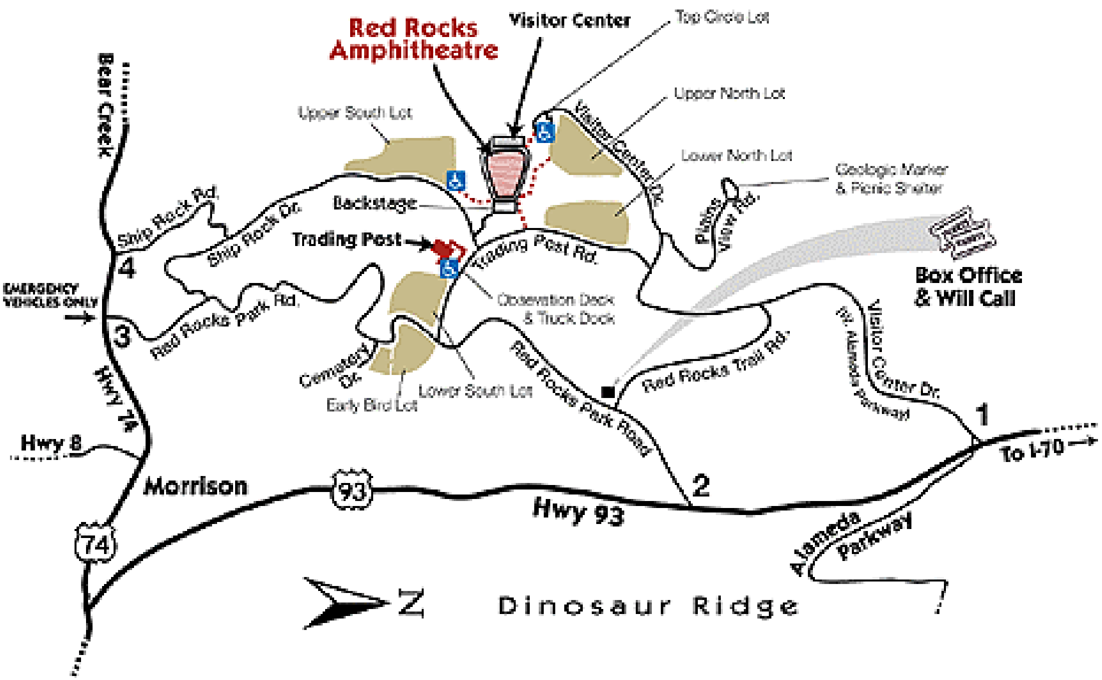

Figure 1. Red Rocks Amphitheatre is located approximately 15 mi west of Denver, Colorado (Image from redrocksonline.com).

GEOLOGY

All of the spectacular red rocks in Red Rocks Park belong to the Fountain Formation, which is approximately 1,000 ft thick at this location (Taylor, 1992). The Fountain Formation is Pennsylvanian in age, approximately 265 to 310 million years old and is composed of thick red beds of sandstones and conglomerates interbedded with thin beds of dark red mudstone. Figure 2 is a photograph of the Fountain beds surrounding the amphitheatre. The sandstones are nonmarine and extensively cross-bedded. The red color of the Fountain formation comes from oxidized iron in the clay cement that holds the clastic grains of the sandstone together. The sedimentary units along the Front Range were turned up at steep angles when the Rocky Mountains formed (Figure 3).

Surficial materials mainly consist of well graded, silty, fine to medium sands with sandstone cobbles and boulders. The site contains many large boulders of sandstone that have merely detached from bedrock or traveled a short distance downslope due to gravity. During excavation, it was difficult to distinguish between some of these large boulders and actual bedrock outcrops. The sizes and depths varied widely, which made it difficult to predict where bedrock would be.

Figure 2. Red Rocks Amphitheatre looking south with sandstone beds of the Fountain Formation dipping eastward in the foreground (Image courtesy of DPL Western History Photo Gallery).

Because of this, each “boulder” needed to be evaluated first to see if it was truly bedrock and then to make sure it was stable.

HISTORY

The Red Rocks park area was recognized as a natural landmark before any settlers arrived in Colorado. Native Americans used this area for shelter against the elements and as protection from enemies approaching from the plains. The base of the amphitheater was used for ceremonies by the Utes and Arapahoes (Mumey, 1962).

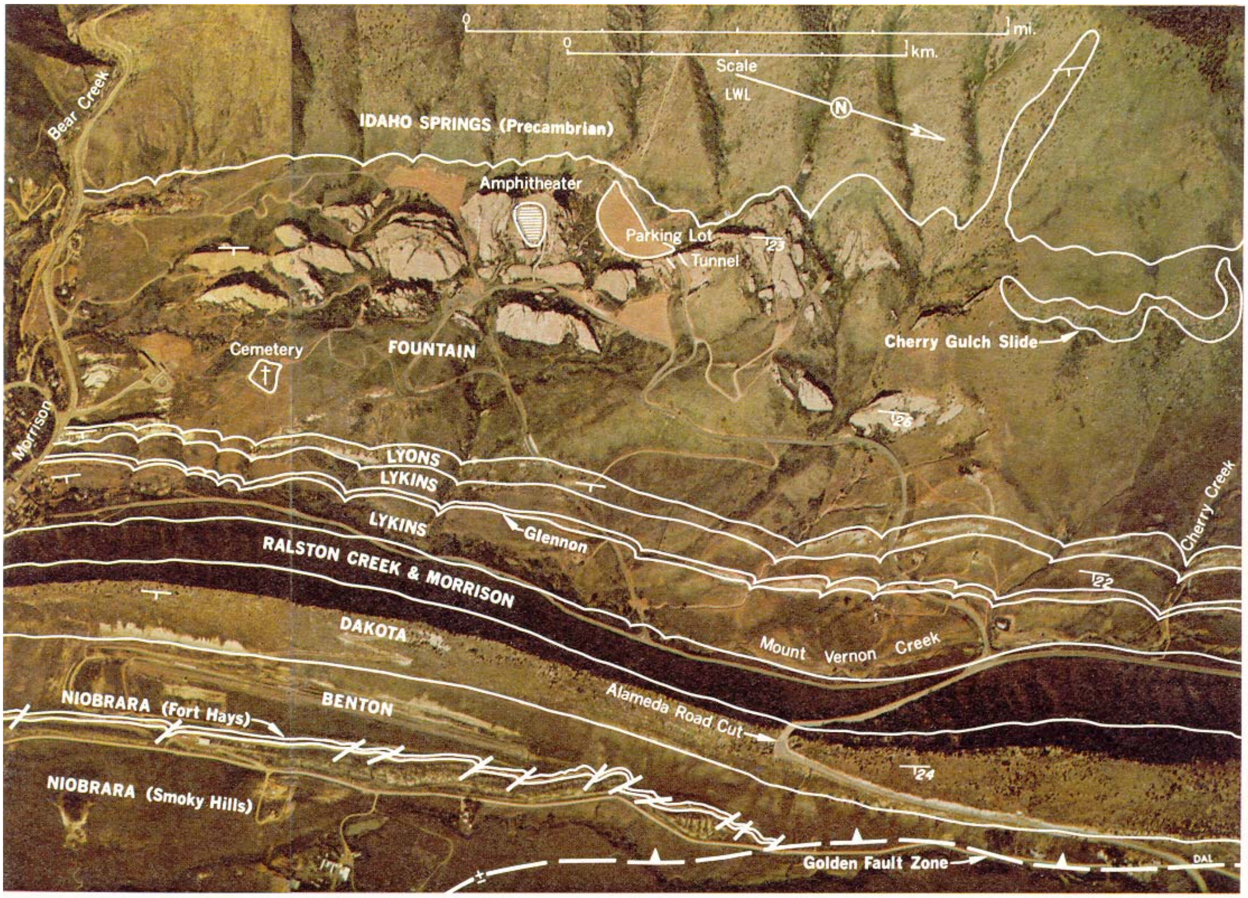

Figure 3. Geologic map superimposed on an aerial photograph of the Red Rocks Parks area (Image from LeRoy, 1978).

In the early 1900's John Walker produced concerts on a temporary platform nestled into the perfectly acoustic surroundings of Red Rocks. The Manager of Denver Parks in 1927, George Cranmer, convinced the City of Denver and Mayor Ben Stapleton to purchase the area of Red Rocks and build on the foundation laid by Walker.

By enlisting the help of the federally sponsored Civilian Conservation Corps (CCC), and the Works Projects Administration (WPA), labor and materials were provided for the venture. Employees of the National Park Service assisted in the construction as well (Figure 5).

From Figure 5, it is evident that end-dumping was practiced at the top of the amphitheater where the new visitor center is located. Approximately 50,000 cubic ft of material was moved. Figure 6 depicts a concrete pour in the amphitheater seating area. The drainage gradient from north to south along the seating slabs is still functioning well today. However, the material underneath these slabs may be eroding.

Denver architect Burnham Hoyt designed the amphitheater with an emphasis on preserving the natural beauty of the area (Figure 7). The plans were completed in 1936, and the amphitheater

Figure 4. Location of the future Red Rocks Amphitheatre circa 1905 (Image courtesy of DPL Western History Photo Gallery).

was dedicated on June 15, 1941, though the actual construction spanned over twelve years from 1936 to 1948.

One of the more challenging aspects of the renovation was the site's historic designation, which required every removed item to be catalogued and reinstalled in its original position. Red Rocks is both a Denver and National Historic Landmark, so any changes are reviewed by the Landmark Preservation Commission (LPC). In 1999, the Friends of Red Rocks (FORR) formed to "enhance the park and amphitheatre while protecting and preserving its historic, architectural, cultural, and natural values." These organizations assist in maintaining the integrity of Red Rocks as an Historic Landmark during any renovation processes.

2001-2003 RENOVATIONS

Renovations were divided into three main areas based on the major construction sequencing and the separation of wall types: (1) concourse (2) visitor center (3) south stairs (Figure 8).

Figure 5. CCC, WPA and National Park Service employees built the original amphitheatre between 1935 and 1947 (Images courtesy of DPL Western History Photo Gallery).

Figure 6. Concrete pour in the amphitheatre seating area (Image courtesy of DPL Western History Photo Gallery).

Figure 7. Original amphitheatre stage designed by Burnham Hoyt (Image courtesy of DPL Western History Photo Gallery).

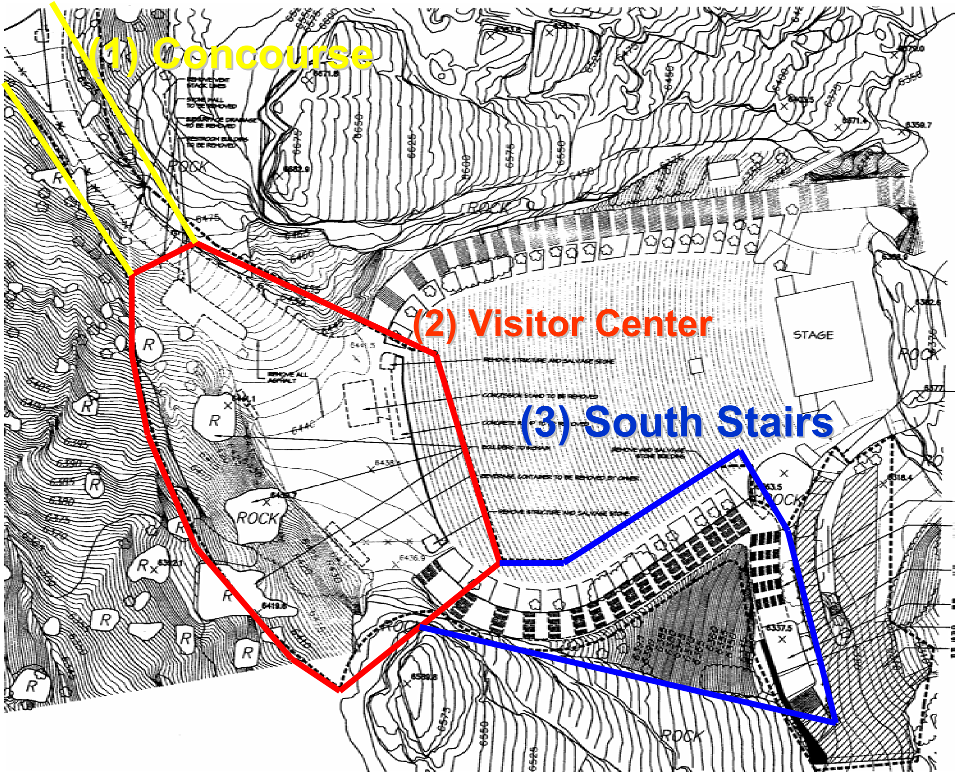

Figure 8. 2001-2003 renovation areas.

In the concourse area, cut walls were designed with soil nails on the northeast side of the new ramp providing better access to the top of the amphitheater and the visitor center. Along the eastern walls of the visitor center, soil nails were also used to provide temporary excavation support as well as permanent reinforcement to the structural walls. An arc-shaped fill wall (MSE) was constructed on the west side of the visitor center to create a foundation for the terrace that looks out at Mt. Morrison. In the south stairs area, ground anchors (tiebacks) were used to support the historic planters and seating area of the amphitheater during the excavation of a new utility trench and installation of a new stairway. Additionally, ground anchors were used to stabilize the slope south of the seating area.

PREVIOUS WORK BY OTHERS

Between 1999 and 2001, several geotechnical investigations were conducted in preparation for the Red Rocks Amphitheatre expansion. The existing conditions investigation (CTC-Geotek, Inc., 1999a) noted drainage problems and void areas underneath the seating areas. A total of approximately 30 borings were drilled, but mostly in easily accessible areas along the entrance ramp to the amphitheatre, in the seating area, and along the stairs. One investigation used a specialty drilling contractor to drill some borings in the south slope adjacent to the amphitheatre seating (Yenter Companies,1999) to perform a preliminary landslide investigation. It was not determined if a slope failure had occurred. A seismic refraction and ground penetrating radar investigation (Geo-Recovery, 2001) was conducted at the top of the amphitheater. Lines were run near the base of the west arch-shaped wall of the Visitor Center where an MSE wall was designed, but the results did not indicate bedrock would be near the surface. A soil profiling investigation by CTC-Geoteck (2001) indicated that voids might be encountered at the top of the amphitheater. After all of these investigations, it appeared as if the on-site materials were highly variable, with bedrock, boulders, natural alluvium, poorly compacted fills, and voids possible at any location during excavation.

CONCOURSE ENTRANCE RAMP

Soil Nail Walls

To make the site more accessible to those with disabilities, the gradient from the parking area to the amphitheater seating needed to be decreased by lowering the elevation of the ramp, which required a 10- to12-ft high cut. Soil nailing was determined to be the most economic shoring method for this location (Figure 9).

Figure 9. The concourse ramp cut was stabilized using soil nails.

The basic concept of soil nailing is to reinforce and strengthen the existing ground by installing evenly spaced steel bars, called “nails”, into a slope or excavation as construction proceeds from the “top down”. This process creates a reinforced section that is itself stable and able to retain the ground behind it. The reinforcements are passive and develop their reinforcing action through nail-ground interactions as the ground deforms during and following construction. Nails work predominately in tension, but can also work in bending/shear in certain circumstances. The effect of the nail reinforcement is to improve stability by increasing the normal force and hence the soil shear resistance along potential slip surfaces in non-cohesive soils; and reducing the driving force along potential slip surfaces in both non-cohesive and cohesive soils. A shotcrete construction facing is applied to the excavated face following the nail installation on each lift to reduce raveling. This process is continued until the final grade is established.

The concourse cut required two rows of soil nails with a 5-ft horizontal by 5-ft vertical spacing in surficial materials. Lengths were directly proportional to the wall height and inclined 15 degrees below horizontal. A waterproofing admixture was added to the shotcrete mix to prevent water infiltration and assist in the drainage improvements to the amphitheatre since the soil nail wall became part of the final interior wall. Details on soil nail wall analysis and design are given below in the section on Soil Nail Walls for the Visitor Center.

Rock and Boulder Stabilization

On-call services were provided to stabilize boulders that were encountered during excavation (Figure 10). Every effort was made to keep boulders in place and preserve the natural landscape at this historic site. Some large boulders were trimmed by controlled blasting when they could not be incorporated into the renovation construction or split and removed. Small boulders that were uncovered during excavation that were not historic features were removed and used for landscaping near the parking lots. Berms were used to keep many boulders stable during construction activities, then base of each one were reburied permanently.

VISITOR CENTER

Soil Nail Walls

Permanent shoring was provided in the form of soil nails along the north and east side of the new visitor center (Figures 11 and 12). The maximum height and length of the soil nail wall was 33 ft and 450 ft, respectively. Along the north side of the excavation, soil nails from the concourse area were continued down slope. Number 8 bars were used in 4-in diameter holes in silty sand materials on a 5-ft horizontal by 5-ft vertical spacing. When weak rock instead of soil was encountered, the drill hole diameter was reduced to 3 inches and the bar size reduced to a #7 with 6-ft horizontal to 6-ft vertical spacing. Soil nails were eliminated when competent bedrock was encountered. The soil nail wall was directly connected to the structural wall of the Visitor Center, so no facing needed to be designed. Adjacent to the seating area, loose silty sand was encountered, requiring longer soil nail lengths to maintain the desired factor of safety. Additionally, the nails in this area were inclined 30 degrees downward in order to avoid intersecting the historic amphitheater seats.

Figure 10. This boulder was uncovered during excavation and needed to be stabilized.

Soil parameters were back calculated for the steep slope on the south side of the amphitheater and from previous work. Estimated friction angles ranged from 16 to 18 degrees and cohesion from 200 to 300 psf for natural surficial material and fill. Estimated bedrock properties were based on typical sandstone values and on unconfined compressive strength tests. Friction angles ranged from 30 to 35 degrees and cohesion ranged from 1450 to 1550 psf for the analysis.

The soil nail wall design consisted of an analysis of the shotcrete facing thickness and reinforcement requirements, the soil nail head and tendon strength requirements, and the pullout resistance of the nails. These parameters were then used to determine the overall stability of the wall using Goldnail (Golder Associates, 1996), a slip-surface, limit-equilibrium slope stability program. A seismic acceleration was selected at 0.025g for this area (AASHTO, 1996). Ultimate bond stresses of 11 psi for soil and 40 psi for rock were used for pullout resistance. Stability analysis resulted in safety factors greater than 1.35 for static and 1.10 for pseudo-static conditions.

Figure 11. Soil nails were used to stabilize the 35-ft high cut during construction of the new Visitor Center at the top of the amphitheatre.

Figure 12. Soil nails were used to support the seating at the top of the amphitheatre during construction (Image courtesy of David Mashburn).

Mechanically Stabilized Earth (MSE) Wall

An MSE wall is a cost-effective soil-retaining structure that can tolerate much larger settlements than traditional reinforced concrete walls, so it is more effective in poor subsoils. By placing tensile reinforcing elements in the soil, the strength of the soil can be improved significantly such that the vertical face of the soil/ reinforcing system is essentially self-supporting. Use of a facing system to prevent soil raveling between the reinforcing elements allows very steep slopes and vertical walls to be constructed safely. In some cases, the inclusions can also withstand bending from shear stresses, providing additional stability to the system.

The MSE wall was form-fitted into the existing geology, leaving two large boulders exposed. The wall extends approximately 270 linear feet and ranges in height from 2 to 35 ft (Figure 13). Geogrid was placed in 1-ft lifts using ¾-in gravel as backfill material to limit deflection. The reinforcement length of the geogrid was 70% of the wall height, except next to the largest exposed boulder (nicknamed “Walter”) where there was only enough space to have reinforcement lengths approximately 40% of the wall height. To compensate for the loss in reinforcement length at this location, 5-ft rock dowels were drilled into “Walter” that were used to anchor the back of the MSE wall to prevent overturning. To prevent the MSE wall backfill material from exerting pressure on Walter and the masonry wall facing, the geogrid was wrapped at both the back and front of the wall. The facing on the MSE wall matches the historic pattern in the amphitheater and was constructed of the same Lyons sandstone blocks as the rest of the amphitheater (Figure 14). The face of the MSE wall is battered at 1 horizontal : 10 vertical back

Figure 13. The MSE wall follows the natural topography in an arc facing Mt. Morrison to the west.

into the wall to allow for some deformation during and shortly after construction without giving the public a toppling feeling when they stand at the top of the wall.

The subsurface material below the MSE wall consisted of fill, colluvium, boulders, and sandstone bedrock. The boulders ranged from 12-in to greater than 30-ft in diameter. Several large boulders were trimmed along the south side of the MSE wall by blasting. Before construction, there was no available geotechnical information regarding the soil parameters of

Figure 14. Construction of the MSE wall with Lyons sandstone facing that matches the pattern and blocks used in other areas of the historical amphitheatre (Image courtesy of David Mashburn).

the foundation material below the MSE wall. A seismic line was offset approximately 20 ft along the alignment of the southern section of the MSE wall, but did not quantify the depth to bedrock below the toe of the wall. At the base of the MSE wall, a rock surface was encountered approximately two feet deeper than expected, so structural fill was used to create a leveling pad to accommodate possible differential settlement.

The MSE wall was designed to satisfy both internal and external stability. The overall global stability of the wall was analyzed using PCSTABL6 (Purdue University, 1999). Soil properties were back calculated using the existing topography assuming a factor of safety of 1.1. Material strengths for the MSE wall itself were assumed to be consistent with a select backfill material with a friction angle of 34 degrees and zero cohesion. External stability analysis resulted in factors of safety for 1.5 for overturning and sliding, and 2.5 for bearing capacity.

SOUTH STAIRS AND CONCESSION

The south part of the seating area was undergoing movement, which raised questions about the slope being a landslide (Figure 15). It appears that damage was being caused by settlement and movement of the fill, which was probably end-dumped into place as uncompacted material over 60 years ago during the original construction of the amphitheatre. Poor storm drainage had contributed to the problem, and large void areas had formed beneath the seats due to erosion and transport of the granular fill material (CGS, 1999).

Figure 15. The south stairs of the amphitheatre and the slope that appeared to be moving before renovations.

Compaction Grouting

Compaction grouting was used to densify the loose fill (Figure 16). Borings were extended to bedrock ranging in depth from five to twenty ft, on a staggered 5-ft pattern in three rows following the imprint of the new stairs. Grout was pumped into the borings until either the grout pressure exceeded 400psi or more than 5 ft^3 of grout was injected per 3-ft interval at a pressure of 100 psi or greater.

Reinforced Soil Slope (RSS)

A reinforced soil slope was constructed over the top of the utilities to form the base of the new stairs and to assist with the stability of the slope below. Due to limited access, material was dumped into place by a crane (Figure 17). One-foot spaced geogrid was wrapped at the face of the slope and at the south side of the stairs.

Ground Anchor (Tieback) Walls

Ground anchors are used to support unstable material in natural slopes or excavations by transferring tensile loads to bedrock (Figure 18). A ground anchor is a high strength steel

Figure 16. Compaction grouting was used to densify fill materials that may have been settling.

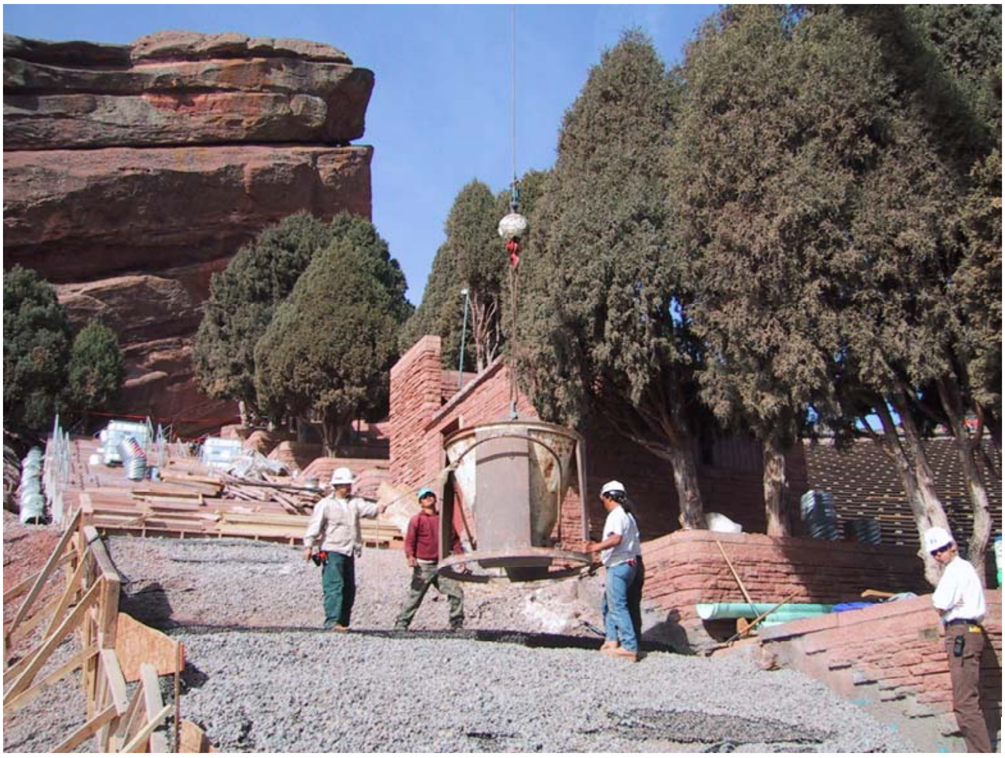

Figure 17. A reinforced soil slope was built as a platform for the new stairs and to assist in the potential instability of the slope below.

Figure 18. Ground anchors were installed to stabilize the south slope and keep the planters along the amphitheatre seating shored during construction.

tendon, fitted with a stressing anchorage at one end and a means of permitting force transfer to the ground on the other end. The anchor tendon is inserted into a prepared hole, grouted in place, and stressed to a specified load.

The stairs on the south side of the Amphitheater were excavated and reconstructed with utilities underneath as well as the addition of restrooms at the bottom of the stairs. Ground anchors were installed in three separate areas of the stairs to accommodate potential movement of the steep slope to maintain the integrity of the planters adjacent to the Amphitheater, and to support the excavation at the base of the stairs. Reinforced soil was installed in the upper portion of the stairs to provide soil reinforcement. In the same area, compaction grouting was performed to densify the loose fill in the slope. In the lower section of the slope, ground anchors were installed with 8-foot by 8-foot panels to stabilize the slope.

At the base of the stairs, two or three rows of ground anchors were required to support the excavated toe of the slope and maintain slope stability during the construction of the new bathrooms and concession stands. Anchors were constructed using 150 Grade #11 epoxy-coated bars with a design load of 126 kips and 50-ft lengths.

Slope stability was analyzed using PCSTABL6 (Purdue University, 1999). Soil parameters were based on the Preliminary Slope Stability Investigation by Yenter (1999). Minimum bond lengths of ten feet into bedrock were specified. Minimum free lengths were determined from the projected location of bedrock in critical cross sections and ranged from 25 to 40 ft.

The overall slope stability analysis resulted in factors of safety above 1.5. The analysis considered slope failures that would impact the south stairs and amphitheater structures. The ground anchors were not designed to mitigate against midslope failures.

SUMMARY AND CONCLUSION

Over sixty years, technology has changed, but our common vision for preserving this unique setting has remained the same. Red Rocks has inspired many geologists and engineers along with the musicians that have performed at this unique venue and, with our corroborative efforts, will continue to inspire people into the future (Figure 19).

ACKNOWLEDGEMENTS

We thank the City of Denver (Theaters & Arenas), the owner of Red Rocks Amphitheatre, and the Sink Combs Dethlefts, the architects for the project, for their support of these renovations and making our work possible. We would also like to thank the Denver Public Library – Western Photo Gallery for allowing us to use photographs from their collection to depict the early days and original construction of the amphitheatre. It was a pleasure working with the entire design and construction team.

Figure 19. Artist's concept of the new concourse and Visitor Center opened in May 2003 to the public (Image courtesy of Sink Combs Dethlefts).

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APPENDIX I: SUPPLEMENTAL READING LIST

Listed below are additional references that were not cited in the text:

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CTC-Geotek, Inc., 1999c, Seating Area Sounding Survey, Red Rocks Amphitheatre, Denver, Colorado: dated June 17.

CTC-Geotek, Inc., 1999d, Soil and Foundation Investigation, Red Rocks Amphitheater Expansion, South Side of Red Rocks Amphitheatre, Denver, Colorado: dated September 14.

CTC-Geotek, Inc., 2000, Soil and Foundation Investigation, Red Rocks Visitor Center, Red Rocks Amphitheatre: dated November 27.

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Federal Highway Administration (FHWA), 1998, Manual for Design and Construction Monitoring of Soil Nail Walls: FHWA Report No.FHWA-SA-96-069R, Washington, D.C., 326p.

FHWA, 1999a, Mechanically Stabilized Earth Walls and Reinforced Soil Slopes Design and Construction Guidelines: FHWA Report No. FHWA-SA-96-071, Washington, D.C., 371p.

FHWA, 1999b, Geotechnical Engineering Circular No.4 Ground Anchors and Anchored Systems: FHWA Report No. FHWA-IF-99-015, Washington, D.C., 185p.

National Park Service, 1935, Grading Plan of Amphitheater, Red Rocks Park, Denver Mountain Parks.

APPENDIX II: WEB SITES

Listed below are web sites that are relevant to this paper:

http://www.redrocksonline.com
http://www.friendsofredrocks.org
http://www.archives.state.co.us/ccc
http://photoswest.org

THE CHALLENGES OF EVALUATING EARTHQUAKE HAZARD IN COLORADO

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Key Terms: earthquake hazard, earthquake risk, seismic hazard, seismic risk, tectonics, building codes, Quaternary faulting

ABSTRACT

In general, Colorado is not considered to be at risk from significant earthquake damage. The state is ranked 30th in the nation in terms of Annualized Earthquake Losses by the Federal Emergency Management Agency (FEMA) and Denver is rated by the U. S. Geological Survey (USGS) National Seismic Hazard maps as having about the same earthquake hazard as Montgomery, Alabama. However, a growing body of data suggests that Colorado may be at greater risk than previously recognized. Colorado has the second largest heat flow anomaly in the North American continent, fifty-eight peaks over 14,000 feet in elevation, and extensive Neogene deformation indicative of an active tectonic province. The catalog of Quaternary faults in Colorado has steadily increased from zero in 1960 to close to ninety in 1998 with many areas of the state unexamined. The strong 1882 earthquake has been definitively located in the northern Front Range. Studies of Quaternary faults in Colorado have resulted in 13 faults being assigned a "maximum credible earthquake" ≥ M 6.25 and as high as M 7.5. With Colorado's rapidly growing population (3rd fastest in the nation), substantially more research needs to be directed toward Colorado's earthquake hazard.

INTRODUCTION

Geotechnical workers face a difficult challenge in assessing earthquake risk in Colorado because, unlike many other states, there has not been a concentrated effort to gather data that can be used to evaluate the hazard. The official categorization of seismic design criteria in the International Building Code (IBC) is based on the USGS' National Seismic Hazard Maps. However, for a variety of reasons, Colorado has been relatively neglected in the gathering of the kind of data that is used in preparing the hazard maps. Because these crucial data sets are incomplete in Colorado, the maps may not reflect the true hazard. Consequently, the geotechnical consultant is commonly placed in the mode of recommending safety on the basis of incomplete data. In Colorado, "No evidence of Quaternary faulting" is *not* the same as "Evidence of no Quaternary faulting."

When one views the entire record of what is known in Colorado about faulting, tectonics, and earthquakes, one is led to the conclusion that caution must be used in blindly following the current hazard categories. Critical facilities should receive a rigorous analysis of the likelihood of a damaging earthquake during their lifetime. "Better safe than sorry", is probably not bad advice for critical-facility design in the western two-thirds of Colorado. However, even the eastern one-third of the state should not be treated lightly because the fault with one of the best-known records of earthquake recurrence in Colorado is located on the plains northeast of La Junta.

One source for information on seismic hazard in Colorado is the geotechnical reports prepared for critical facilities. Commonly, these studies determine a Maximum Credible Earthquake (MCE) for faults that might generate earthquakes affecting the site under study. A compilation of Maximum Credible Earthquakes (MCE) ≥ M 6.25 that were assigned to various faults in Colorado portrays a sobering picture (Figure 1).

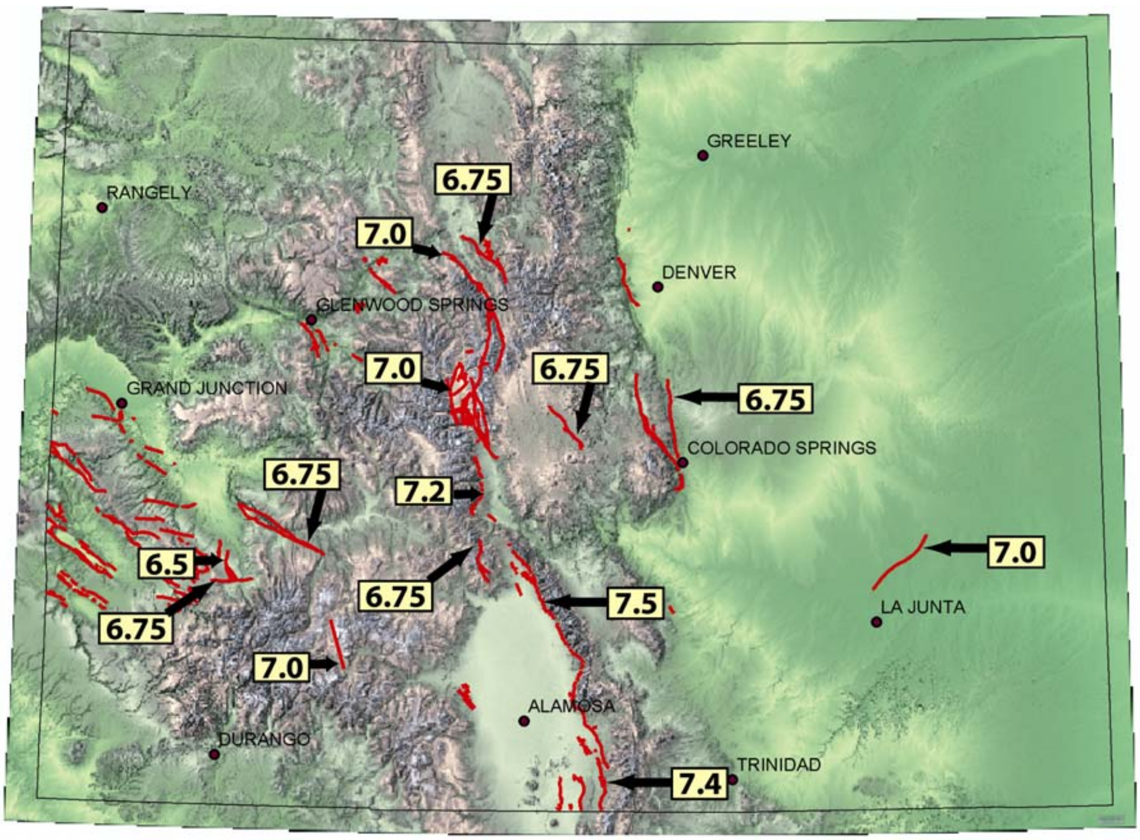

Figure 1. Known Colorado Quaternary Faults and Maximum Credible Earthquakes (MCE). The faults (red) and MCE values are from Widmann, and others (2002) and Frankel, and others (2002). MCE values exceeding M 6.25 were extracted from geotechnical reports and publications individually referenced in Widmann, and others (2002).

INTERMOUNTAIN WEST (IMW) SEISMIC AREA

Colorado is part of the InterMountain West (IMW) seismic area. In the IMW, an extensional tectonic environment began in the Miocene and continues today (Hamilton, 1989). All of the seven IMW states have evidence of Quaternary faulting (Frankel, and others, 2002) and all have experienced basaltic volcanism during the past 4,200 years. And, all but two (AZ & NM) have experienced earthquakes M > 6.0 within their borders during the last century and a half (Stover and Coffman, 1993).

The IMW is often compared to California when considering seismic activity, fault slip rates, GPS budget, and earthquake hazard. Obviously, the IMW states pale in these comparisons with ultra-active California (as do all other states with the possible exception of Alaska). However, when one compares the tectonic characteristics of the IMW to the Central and Eastern United States (CEUS) seismic area, the IMW characteristics do not seem nearly as insignificant as when compared to California. The eastern seaboard has been a passive margin since the opening of the Atlantic in the Triassic. Yet, South Carolina alone has a high-hazard area that is half the size of the high-hazard area in the six states of the IMW, even though the IMW is seventeen times as large as South Carolina (Figure 2).

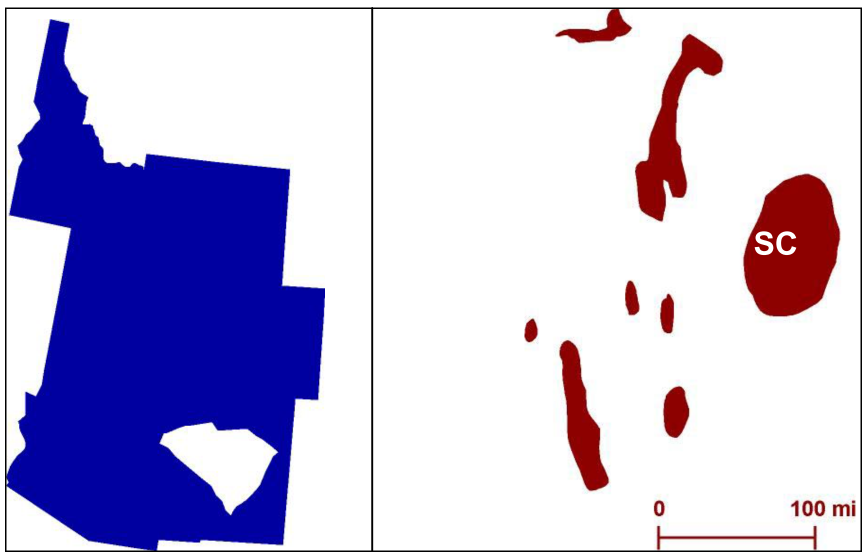

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Figure 2. Comparison of earthquake hazard in the IMW and South Carolina. a.) Comparison showing that the area of the six IMW states is 17 times greater than the area of South Carolina [white inset] b.) The large blob on the right is the high-earthquake-hazard area in South Carolina and the other red blobs are the high-earthquake-hazard areas (Frankel, and others, 2002) in the six-state IMW (http://geohazards.cr.usgs.gov/eq/html/us2002oct.htm).

OFFICIAL CATEGORIZATION OF HAZARD AND RISK

Earthquake *hazard* maps relate to the probability of a particular site undergoing a given level of ground acceleration caused by an earthquake. *Risk* maps add the dimension of exposure of human life, as well as the design and value of buildings to the equation. An area could be considered to be a high, earthquake-hazard area, but a low risk because no one lives within the area. The epicenter of the 2002, M 7.9 Denali earthquake in Alaska is an excellent example of a high-hazard, low-risk area.

New York City, NY and Santa Rosa, CA provide excellent examples of the difference in hazard and risk. New York City's earthquake hazard (peak ground acceleration) is 15 times lower than Santa Rosa's hazard. But, New York has a higher population, higher building stock value, and lower earthquake-resistant design than Santa Rosa. Therefore, FEMA's calculation of earthquake risk gives New York City annualized earthquake losses of $56 million versus Santa Rosa's $51 million.

Two maps relate to earthquake hazards and risk in Colorado. The National Seismic Hazard Maps (Figure 3) created by the USGS form the underpinning for the risk maps (Figure 4) created by the FEMA. The hazard maps also provide data for calculations used in seismic-design formulae of the International Building Code (IBC).

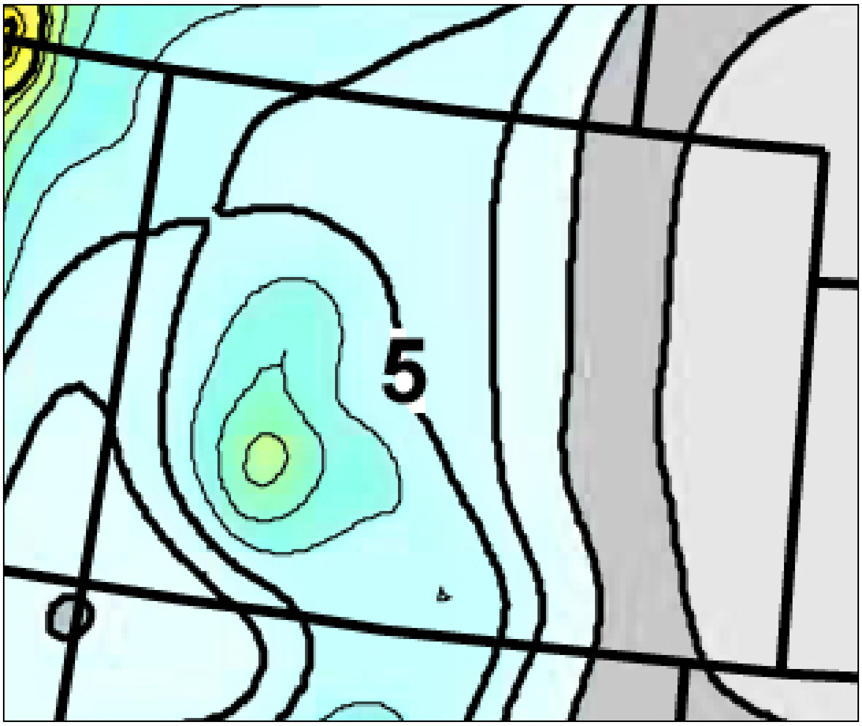


Figure 3. Colorado Seismic Hazard. Peak Acceleration (%g) with 10% Probability of Exceedance in 50 Years. Excerpted from the 2002 National Seismic Hazard Maps (USGS). The contour lines indicate the values of peak ground acceleration measured as a percent of the force of gravity (g). These maps indicate that peak ground accelerations of .08 to .09g are the maximum expected anywhere within the state at the 10% probability level during a 50 year period. Five additional maps at various probabilities and spectral and peak accelerations are available online at http://geohazards.cr.usgs.gov/eq/html/natlmap.html.

2002 National Seismic Hazard Map (USGS)

National Seismic Hazard Maps are prepared by the USGS and updated every five years. The 2002 series are the most recent release and depict probabilistic ground motions. A team of USGS seismologists and geologists evaluate data throughout the United States (Frankel, and others, 2002). Regional workshops provide an opportunity for stakeholder input during the draft

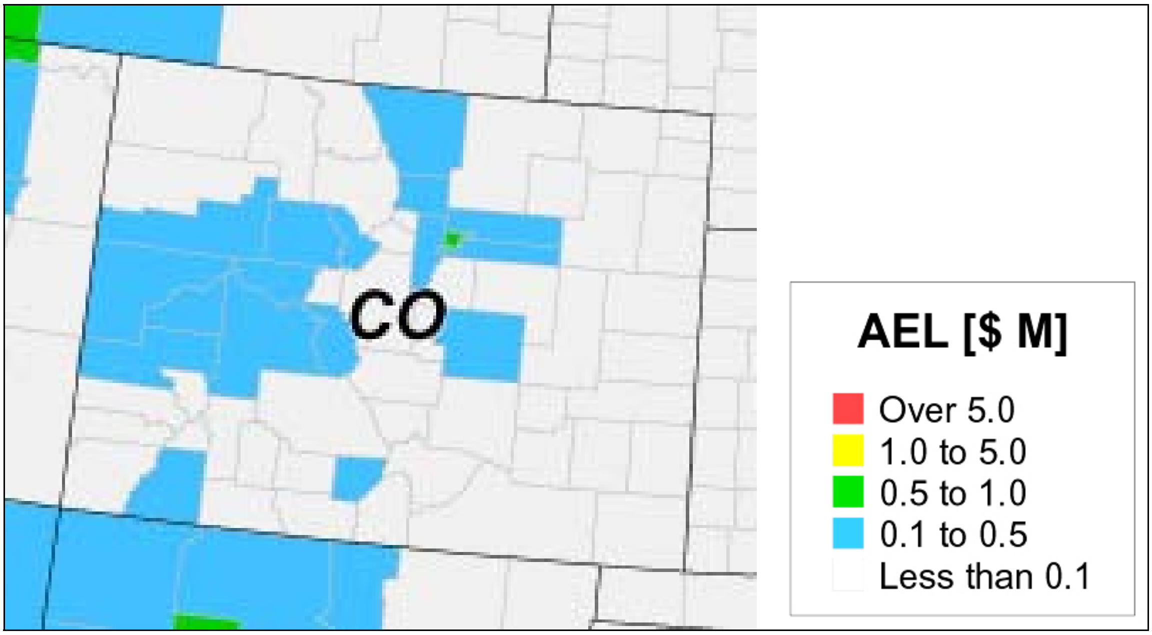


Figure 4. Annualized Earthquake Losses. The analysis of earthquake risk in Colorado performed by FEMA in HAZUS99 indicated Annualized Earthquake Losses (AEL) for the state of $5.8 million distributed by county as shown above. The full report is online at http://www.fema.gov/hazus/eq_ael.pdf.

process. The currently posted maps depict peak ground acceleration and 0.2 sec and 1.0 spectral acceleration with 10% and 2% probabilities of exceedance (PE) in 50 years. Additional maps will eventually be posted.

Factors that are considered in preparing the maps are:

- Historical seismicity— b value
- Faults slip rates— >0.2 mm/year.
- Quality factor (Q)— the ability of the lithosphere to attenuate seismic waves.
- Site amplification— firm rock or hard rock.

The results are presented in a variety of ways. Eventually a set of 12 maps of the U. S. will show contour lines depicting various levels of probabilities, ground acceleration, and spectral periods, e.g.:

Peak Acceleration (%g) with 10% Probability of Exceedance in 50 years
Peak Acceleration (%g) with 2% Probability of Exceedance in 50 years
Peak Acceleration (%g) with 5% Probability of Exceedance in 50 years
0.2 sec Spectral Acceleration (%g) with 10% Probability of Exceedance in 50 years
0.2 sec Spectral Acceleration (%g) with 5% Probability of Exceedance in 50 years
0.2 sec Spectral Acceleration (%g) with 2% Probability of Exceedance in 50 years
0.3 sec Spectral Acceleration (%g) with 10% Probability of Exceedance in 50 years
0.3 sec Spectral Acceleration (%g) with 5% Probability of Exceedance in 50 years
0.3 sec Spectral Acceleration (%g) with 2% Probability of Exceedance in 50 years

1.0 sec Spectral Acceleration (%g) with 10% Probability of Exceedance in 50 years
1.0 sec Spectral Acceleration (%g) with 5% Probability of Exceedance in 50 years
1.0 sec Spectral Acceleration (%g) with 2% Probability of Exceedance in 50 years

2000 National Risk Assessment (FEMA)

In September of 2000, FEMA released a national study of earthquake risk using their risk analysis model, HAZUS99. The evaluation showed a risk of Annualized Earthquake Losses (AEL) of $4.4 billion for the nation and $5.8 million for Colorado. Colorado ranked 30th in the nation behind such states as Ohio, Virginia, North Carolina, Georgia, Delaware, Alabama, Pennsylvania, New Hampshire, Connecticut, and New Jersey.

In addition to these probabilistic evaluations, the model can also be used to conduct damage evaluations for deterministic earthquakes. According to FEMA, "Once the size and location (epicenter) of a hypothetical earthquake is selected, the HAZUS software, using a series of mathematical formulas, calculates the violence of ground shaking, the amount of damage, the number of casualties, the number of people displaced by damaged structures, and the disruption and economic losses caused by the earthquake. These formulas describe the relationship between earthquake magnitude, violence of ground shaking, building and utility system damage, cost of repair, and indirect economic impact. HAZUS allows for changing the size and location of the hypothetical earthquake to see the range of damage that may occur to the community."

In cooperation with FEMA's Region VIII, the Colorado Geological Survey (CGS) conducted two deterministic evaluations. The results indicate that a repeat of the 1882, M 6.6 earthquake north of Estes Park would cause $240 million in losses. An evaluation of the effects of a M 5.8 earthquake in the vicinity of the 2001 Trinidad earthquake swarm would result in $15 million in losses. HAZUS99 is a very effective tool for evaluating the potential losses from an earthquake of a given magnitude in a given location.

BUILDING CODES

Until the year 2000, those Colorado municipalities who chose to adopt a building code drew from the Uniform Building Code (UBC) that was updated every three years; most recently in 1997. In 2000, the International Building Code (IBC) replaced the UBC. Both the UBC and IBC have requirements of earthquake-resistant designs for buildings.

International Building Code (IBC)

Prior to 2000, at least three groups in the United States issued building codes. Denver used the 1997 UBC that divided the country into six Seismic Zones: 0, 1, 2A, 2B, 3 & 4 each with its own seismic-design criteria. [The higher the Zone number, the more stringent the seismic design criteria.] Most of Colorado was in Zone 1, requiring only minimal structural detailing requirements. The eastern 15 percent of the state was in Zone 0, on a par with Minnesota. The boundary between Zones 0 and 1 passed well east of the Front Range Urban Corridor, all of which was included in Zone 1. A small part of southern San Luis Valley was put in Zone 2B.

Recently, the three code councils merged into The International Code Council and now issue one combined code: the IBC. The IBC no longer issues the zone maps. Rather, the seismic design part of the IBC uses formulae to calculate required levels of design based on data from the USGS National Earthquake Hazard Maps that show contours for varying levels of ground acceleration at different periods and probabilities.

IBC versus UBC and Denver's Solution — A thorough review of the seismic-design implications of the IBC 2000 for Denver is given by Jackson (2001). He illustrates how the IBC 2000 actually reduces the seismic design criteria over the UBC 1997 for Front Range buildings founded on very dense soils and rock. And, he further illustrates how the determination of Seismic Design Category (SDC) varies up and down the Front Range. Following the IBC strictly would require Lakewood to have higher seismic-design criteria than the City of Denver. The ground acceleration map contours from the IBC generally decrease going east from the Front Range, so that communities such as Aurora, Greeley and most of Denver would be allowed to design buildings for Seismic Design Category A, using only 1% of gravity loads for the equivalent lateral earthquake force. This is significantly lower than the design of most Front Range buildings under previous codes.

Because the net effect of the IBC 2000 was to reduce the seismic-design criteria of the UBC 1997 for Denver, the City of Denver IBC 2000 Structural Sub-committee felt that it was imprudent to lower the seismic-design requirements. This committee, composed of City structural engineers and representatives from the Structural Engineers of Colorado recommended that the IBC 2000 adoption by Denver preclude the use of SDC A, thereby maintaining approximately the same seismic design criteria as provided for in prior codes. A general review of the 2000 IBC seismic provisions compared to the 1997 National Earthquake Hazard Reduction Program(NEHRP) provisions can be found online at http://www.skghoshassociates.com/Comparisons%20of%20seismic%20codes.htm.

Adoption of Building Codes — A first step in safe building practices is to get government jurisdictions to adopt modern building codes. In contrast to 40% of the states, Colorado does not have statewide building code requirements. Thus, adoption of building codes is spotty throughout the state. As of January 31, 2003, only six counties and 19 municipalities in Colorado have adopted the 2000 IBC (http://www.icbo.org/).

Building Code Enforcement — Adoption of a building code is only the first step. The code must then be enforced to be effective. Here again, Colorado is lacking in enforcement of even the minimal, seismic-design criteria in existence.

In addition to structural design criteria, the UBC and IBC require that non-structural mechanical and electrical systems (heating, ventilation, and air conditioning units, boilers, fans, cooling towers, and similar equipment) must be restrained in order to prevent being shaken loose or toppling over during a moderate earthquake. Bonkoski and others (2000) polled building inspectors along the Front Range to determine whether these provisions were being enforced. All of the responding inspectors indicated they were aware of this section of the code, but 80 percent of the respondents indicated that they do not enforce it, and 60 percent responded that they do not feel that this section of the code is necessary.

DATA PROBLEMS IN COLORADO

A variety of data deficiencies create a less-than-comforting situation for those charged with the responsibility of making evaluations of earthquake hazard in Colorado.

Earthquakes

Many factors contribute to concern about the validity of Colorado's earthquake record. The historical record is short. The lack of a modern seismometer network makes it difficult to locate and detect earthquakes. The lack of knowledge about attenuation of earthquakes in Colorado makes it difficult to predict the strength of ground acceleration. Additionally, the existence of induced earthquakes from fluid injection complicates the attempt to sort these from natural earthquakes.

Historical record of earthquake activity — One of the drawbacks with Colorado's seismic record is the same as that of most of the IMW states: a record of less than 175 years compared to 400 years in the CEUS. Since 1867, the historical record includes more than 500 earthquakes (Kirkham and Rogers, 2000 and online NEIC data through 2003). Charlie and others (2002) analyzed the earthquake catalogue and concluded that it contains 137 independent natural earthquakes between 1867 and 1996. They tested the completeness of the earthquake record and determined that their 137 independent earthquakes are complete for ML ≥ 4.0 between 1870 and 1950, ML ≥ 3.5 between 1950 and 1960, ML ≥ 3.0 between 1960 and 1970, and ML ≥ 2.5 since 1970. The record through 2002 includes fifteen earthquakes of intensity VI or greater. On average, earthquakes causing MM Intensity ≥ V occur about every four years (Figure 5). During a recent six-month period in 2001-02, Colorado experienced four earthquakes M ≥ 4.0.

The strongest earthquake in Colorado during the past century-and-a-half was M_w 6.6. This 1882 earthquake frightened people in Denver and other northern Front Range cities. It was so strong that the bolts holding the electric generators for Denver were snapped off and power was knocked out. The epicenter of the earthquake was uncertain for over a century. However, careful research by CGS scientists in 1986 determined that the earthquake was centered about ten miles north of Estes Park (Kirkham and Rogers, 1986). Research by USGS scientists (Spence, and others, 1996) confirmed this conclusion (Figure 6). Two other reviews affirmed that the location was in the northern Front Range (Stover and Coffman,1993; Bollinger,1994).

Evidence of stronger past earthquakes can be determined by offsets of recent geologic deposits in trenches across active faults. Study of deposits in Colorado show that magnitude 7.0 or higher earthquakes probably occurred on several faults since humans have been living here.

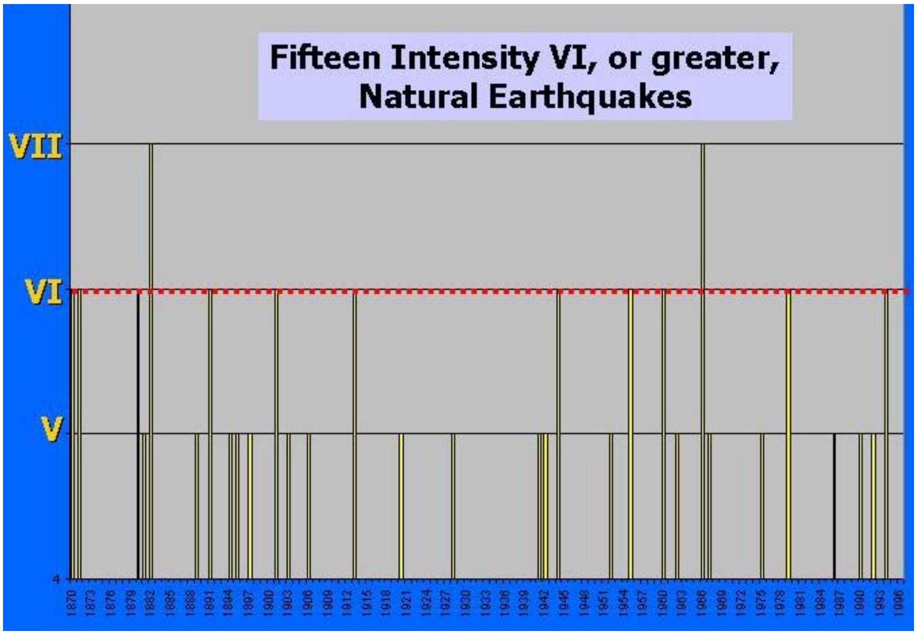


Figure 5. Naturally-occurring earthquakes of Modified Mercalli Intensity V in Colorado from 1870-1996. Intensity VI includes such effects as– People have trouble walking. Objects fall from shelves. Pictures fall off walls. Furniture moves. Plaster in walls might crack. Trees and bushes shake. *Data from* Kirkham and Rogers, 2000.

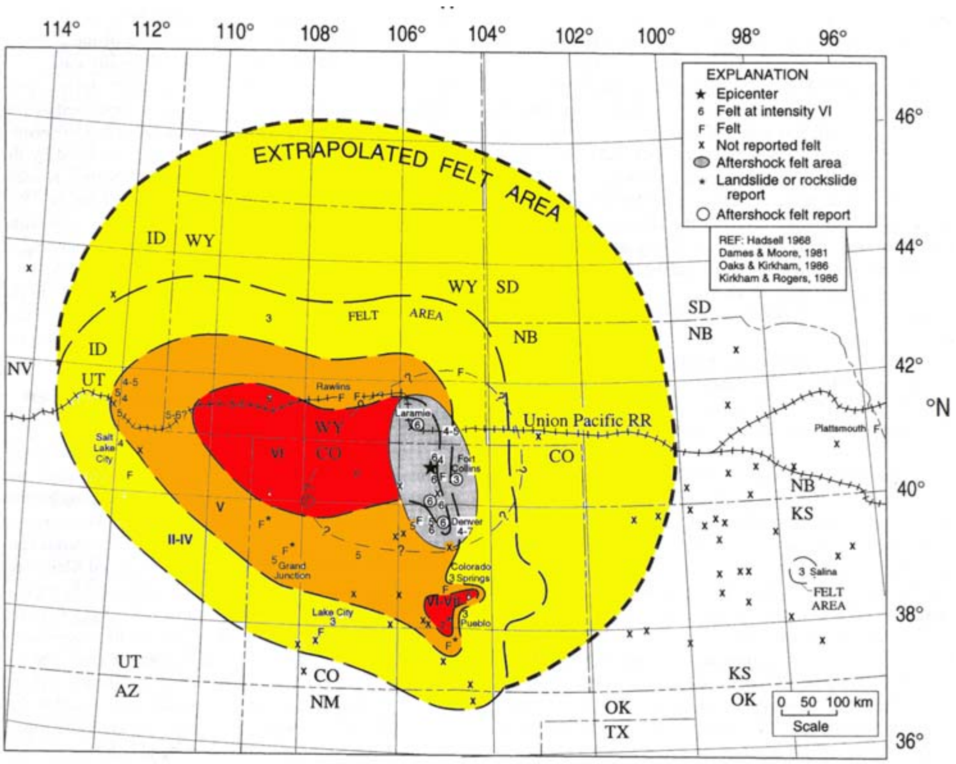
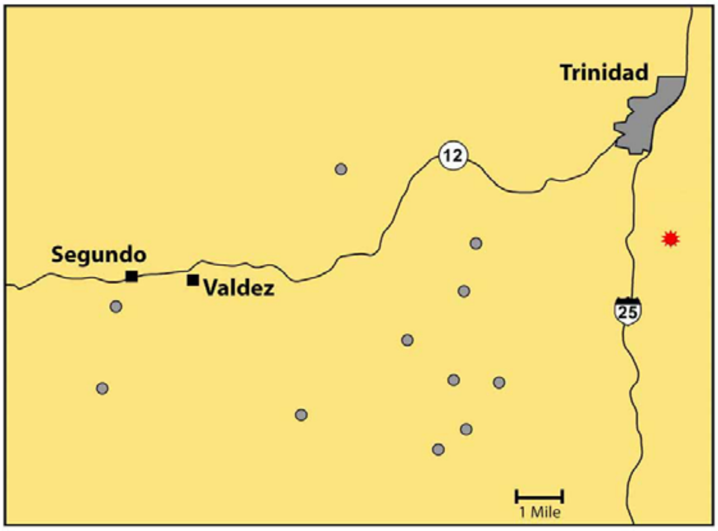


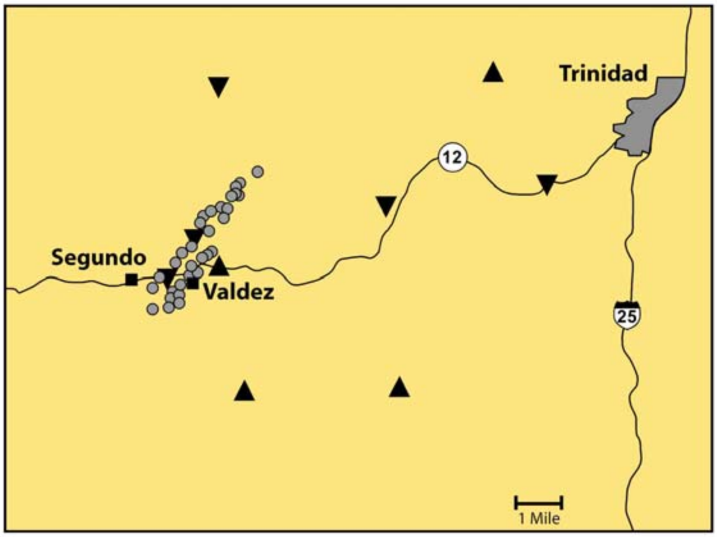
Figure 6. Isoseismal map of the 1882 Mw 6.6 =/-0.6 earthquake. Red contours show area of Modified Mercalli Intensity VI & VII. Gray shaded area shows felt area of aftershock. From Spence and others, (1996)

Difficulty in detecting and locating earthquakes — Until the summer of 2002, Colorado had only two seismographs as part of the National Earthquake Information Center (NEIC) network. Because they were so close together and one of them was in a very noisy location, we effectively had only one station within the state. This situation makes it difficult to detect and precisely locate smaller earthquakes.

The Trinidad swarm of 2001 vividly illustrates the problem of locating earthquakes (Figure 7).



a.



b.

Figure 7. Epicenters of the 2001 Trinidad Earthquake Swarm. a.) Locations of earthquakes reported by the NEIC prior to installation of the local network. The earthquakes appear to be random and are scattered over 75 square miles. The largest earthquake, M 4.6, was calculated to be two miles south of Trinidad (red dot). b. Tight northeast-southwest cluster of earthquake locations determined with the USGS local network. Portable seismographs shown by triangles, earthquakes shown by circles. *Modified from* Meremonte and others, 2002.

The largest earthquake of the swarm was a magnitude 4.6. Its location was initially reported as two miles south of Trinidad. However, Trinidad reported no damage. CGS geologists discovered Modified Mercalli Intensity VII damage in Segundo and Valdez, 11-12 miles west of the reported earthquake location. Pictures were thrown off walls, plaster was broken, bottles were emptied out of cabinets, and a chimney was broken and thrown into the street. The USGS quickly deployed a dense network of portable and temporary seismographs to better understand the earthquakes (Meremonte and others, 2002). Studies using the well-located earthquakes revealed that the largest earthquake was actually located under Segundo, more than ten miles west of the initial location report near Trinidad. Several lines of evidence also showed a good correlation of the earthquakes with the projection of a fault exposed at the surface (Matthews and Morgan, in preparation).

Fortunately, the USGS has recognized the problem of accurately locating earthquakes in Colorado and has funded the installation of two permanent, modern seismographs in the state that will be part of the ANSS national network. One went online at the Great Sand Dunes National Park in July of 2002. The second is scheduled for Kit Carson County. This is an important step toward a better understanding of which faults in Colorado are currently generating earthquakes. Also, an analysis for local earthquake events in the PASCAL data set was recently funded by NEHRP and is currently underway by Dr. Anne Sheehan at the University of Colorado.

Attenuation of Earthquakes — The Quality Factor (Q) indicates how an area dampens seismic waves; higher Q values dampen less, lower Q values dampen more. The CEUS seismic area has a higher Q value relative to California's. Therefore, earthquake waves are considered to be reduced less in the CEUS causing shaking over a wider area than a similar-sized earthquake in California would cause.

The Q value for Colorado is unknown. In the National Seismic Hazard Maps an assumed Q is used. The CEUS Q value was assumed for most of Colorado in the 2002 hazard maps, except for the San Luis Valley where the attenuation value for the Western United States (WUS) was used. In the hazard maps, the Q for a given earthquake is assigned according to which attenuation area the earthquake epicenter is located. Because the boundary between the CEUS and WUS attenuation zones enters Colorado, an interesting dilemma arises.

The San Luis Valley is bounded on the east by the Holocene, Sangre de Cristo fault (McCalpin, 1982) and is the only part of Colorado in the WUS attenuation zone. Therefore, although the Q between Denver and the Sangre de Cristo fault is the higher CEUS value, a lower WUS value would be assigned to an earthquake occurring on the Sangre de Cristo fault because it originated barely within the WUS attenuation boundary. This has the effect of lowering the forecast shaking in Denver from a strong earthquake on that active fault.

Correlation of microseismicity with faults — Several studies have shown clustering of microseismicity on specific faults in Colorado (Sheehan, 2000; Sheehan, 2003, this volume; Godchaux, 2000; Matthews and Morgan, in preparation). However, this relationship is still poorly understood throughout much of the state because of the absence of a complete fault catalogue. At this point in the state of our knowledge, it is probably imprudent to assert that microseismicity in Colorado is not related to specific faults.

Induced versus natural seismicity— Many earthquakes catalogued in Colorado (Kirkham and Rogers, 2000) are considered to be induced by fluid injection at either the Rocky Mountain Arsenal, Rangely Oil Field, or in the Paradox Valley (Charlie and others, 2002). Determining whether earthquakes are natural or human-induced can be problematic such as in the Trinidad swarm (Meremonte, and others, 2002; Matthews, 2002) and at the Rocky Mountain Arsenal (Frankel and others, 2002). Construction or mine blasts are much easier to sort out because of their unique first-motion patterns.

Paleoliquefaction

Paleoliquefaction features (seismites) provide important information for the National Seismic Hazard Maps in areas such as South Carolina, Illinois, Missouri, Washington, and California. Liquefaction seems more likely to occur in humid environments than arid or semi-arid environments such as Colorado's. However, paleoliquefaction features have been described in arid Death Valley. A concentrated search for paleoliquefaction features has not been made in Colorado, but suspicious features are beginning to turn up. Many areas in Colorado have been identified that contain conditions suitable for liquefaction, i.e., groundwater table < 40 feet and unconsolidated sediments.

Faults

Recognizing faults and dating their movement in Colorado is particularly challenging. Colorado's claim to one of the largest expanses of Precambrian crystalline rock (Noe and Matthews, this volume) in the Western U.S. makes dating movement on faults in the mountainous areas exceedingly difficult. Young strata are commonly stripped by erosion from these areas leaving only rocks that are more than a billion years old on each side of a fault. Obtaining slip rates on faults in environments that are not particularly amenable to creating and/or preserving datable strata is also difficult. Because of the uncertainty involved in dating movement on these faults and because of the lingering skepticism about the level of seismicity in Colorado, a higher standard of proof is sometimes applied than in areas such as California and Washington.

Recognizing the existence of faults — Much of Colorado's tectonically active terrane exposes Precambrian crystalline rock. Published mapping of faults in these areas is irregular. Morgan (2003) digitally compiled all of the published faults in the Front Range. His maps clearly illustrate that adjacent maps have vastly different patterns and intensity of faulting depicted (Figure 8). Geotechnical workers must be wary of relying on published maps to define the faults in an area.

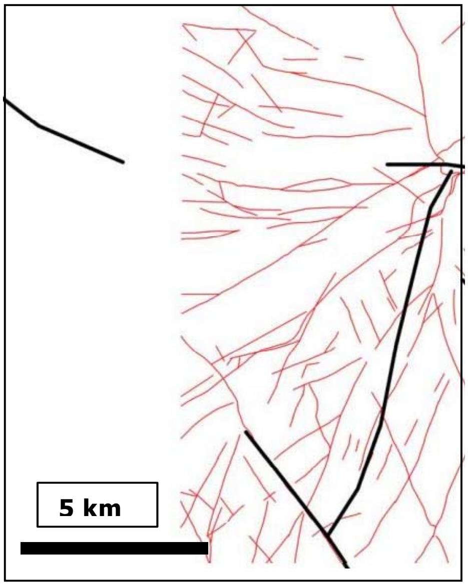


Figure 8. Comparison of faulting at different scales. This map shows published faulting along the eastern flank of the Front Range from Morgan, 2003. Red faults are from 1:24,000 mapping and black faults are from 1:250,000 mapping. The vertical line where the red faults appear cut off are 7.5 minute quadrangle boundaries. Note that there is only one fault from the 1:250,000 map shown west of that line. Yet the true density of faulting west of that cutoff is probably the same as mapped in the east.

Quaternary Faulting — The current catalogue of Quaternary faults and folds in Colorado includes 92 faults and 6 folds (Figure 9). However, the National Earthquake Hazard Maps include only four of these faults in their calculations. More of Colorado's faults are not included because they lack published evidence of slip rates > 0.2mm per year. Some faults that do have published slip rates > 0.2 mm were not included in the newest hazard maps because USGS and CGS geologists concurred that further documentation was required before making a decision on whether to include them in the calculations of hazard.

Dating fault movement — Much of the mountainous terrane in Colorado is composed of Precambrian crystalline rocks in excess of one billion years in age. Dating a fault in this terrane is problematic for determining earthquake hazard. All that is immediately obvious is that the fault has moved sometime since the Precambrian rocks cooled. If the topography appears to be at different elevations on either side of the fault, it is probably justified to try and determine whether there is evidence that can rule out the possibility that it has moved in recent geologic time. Because the catalogue of young faulting in Colorado continues to grow, it becomes questionable whether one can pronounce a fault safe to build upon in the absence of defensible evidence. In the absence of evidence that a fault has not moved in the Quaternary, should one declare that it is safe to build on? This question is further discussed in the section below on Colorado: An Active Tectonic Area.

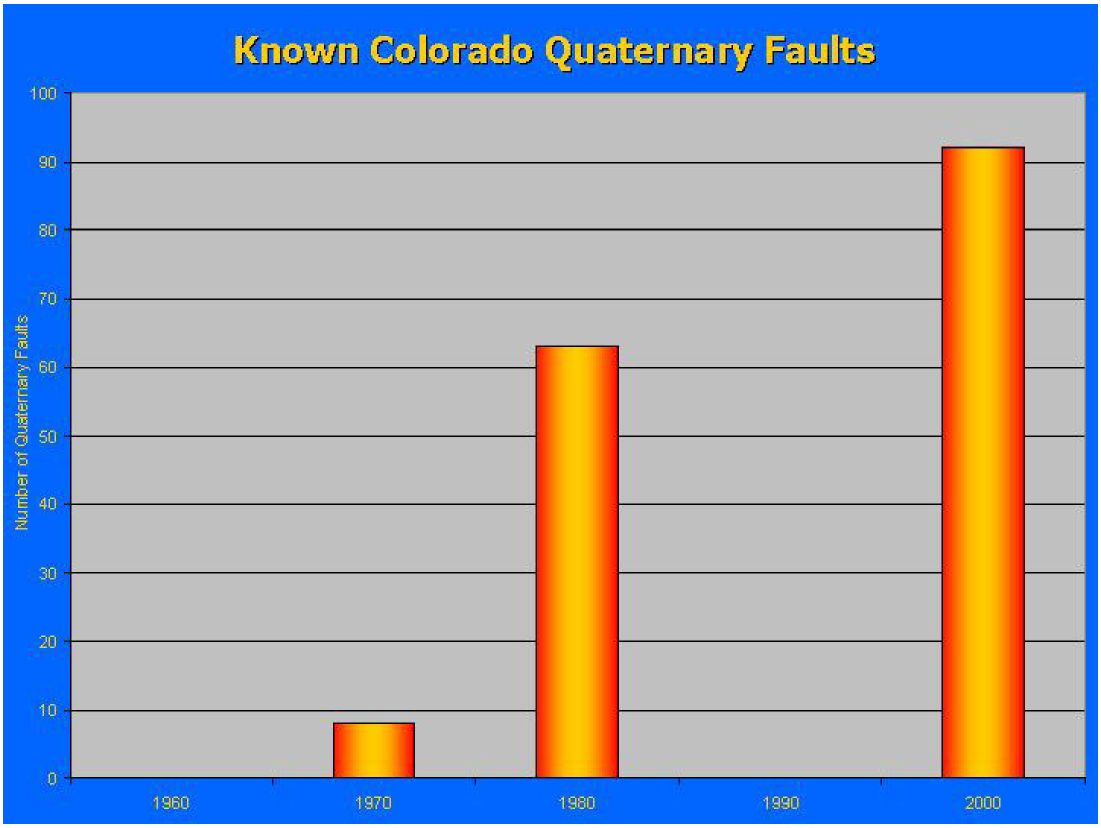


Figure 9. Growth in the Number of Known Quaternary Faults. In 1970 our catalogue of Quaternary faults totaled eight (Scott, 1970). By 1980 the number of identified Quaternary faults increased to more than 60 (Kirkham and Rogers, 1981). The most recent catalogue of Quaternary faults totals 92 (Widmann and others, 1998).

Recurrence Intervals and slip rates— Even where evidence of displacement of Quaternary strata is present, it is difficult to get the data required for slip rates or return periods for large earthquakes. Defensible slip rates require an exposure (natural or mechanically trenched) that shows faulted strata that are correlateable and dateable. Such data are hard to obtain in Colorado.

The Cheraw fault northeast of La Junta has one of the better records of recurrence found to date in Colorado. Yet, even it is somewhat of a fluke. The Cheraw fault trends northeast-southwest and has a subtle scarp facing northwest opposing the regional drainage gradient to the southeast. As a result, ponding occurs at the base of the northwest-facing scarp. These ponds create organic rich sediments that can be correlated and can be dated with radiocarbon methods. Crone and others (1997), trenched the fault and determined that three strong earthquakes occurred on this fault during the past 22,000 years. The results gleaned from this study met the criteria for the hazard maps and the fault was included in the 2002 National Seismic Hazard Maps. However, workers who have studied this fault believe that if the fault dipped to the southeast rather than the northwest, the a scarp would probably not be preserved and the fault would most likely never be discovered. In the unlikely event that such a fault were discovered and trenched, there probably would be no correlative and dateable strata because conditions would not exist to create the ponding and associated dateable, organic sediments.

Because Colorado is such an active area, erosion is more common than deposition in the mountainous areas. This creates a paucity of young deposits useful for evaluating earthquake hazards. As Steven (2002) states, "… erosion has been the dominant geologic process acting on the Southern Rocky Mountains during the late Cenozoic, and by its nature, erosion progressively destroys the history of its own evolution". Many of the young deposits that do exist, are coarse clastics that don't make for easy correlation and dating when they are faulted. As a result, very few slip rates have been obtained for Quaternary faults in Colorado and the few that have been reported are often challenged as not being sufficiently definitive. Documentation of recurrent faulting has been achieved on the southern Sawatch fault (Ostenaa, and others, 1981) and on the Sangre de Cristo fault (McCalpin, 1982) which qualifies them to also be included in the 2002 National Seismic Hazard Maps.

Default Soil Classification

Another problem with assessing earthquake risk through HAZUS99 modeling in Colorado is the lack of good compilations of soil types. Because of this lack of data which states like California have gathered, HAZUS just assumes a default soil type in Colorado. Data on soil types must be compiled before useful "shake maps" can be generated and before the most meaningful results can be obtained from HAZUS.

COLORADO: AN ACTIVE TECTONIC AREA

The dogma being taught in most Colorado universities until the early 1970s was that all of the most recent faulting occurred during Laramide mountain building (~80 to 40 m.y.a.). The only post Laramide activity was considered to be broad regional warping with no faulting, i.e. "the faults have all been dead for 40 million years." Research presented in the early 1970s (Curtis, 1975) set that notion on its ear by documenting significant and widespread post-Laramide faulting.

It is somewhat naive to suggest that a state with 58 peaks over 14,000 feet high, and the highest average elevation in the country (6800 feet above sea level), does not have active mountain building going on. The notion that these mountains are just unroofed remnants of the Laramide mountains, or are only gently upwarped over a broad area, is not substantiated by the data. Rather, they were uplifted by thousands of feet of movement along faults in the past 25 million years, much of it in the past five million years (Steven (2002). Holocene faulting and volcanism, high heat flow, earthquakes, and rugged, challenging mountains indicate that this activity continues today.

Heat flow and volcanism — Heat flow is one common indicator of active tectonism. Colorado has the second-largest, high-heat-flow anomaly in North America (Blackwell and Steele, 2000). The state has 93 large hot springs with hundreds of smaller, hot springs (George, 2000). Central Colorado is also underlain by low-velocity, mantle material (Lerner-Lam, and others, 1998; Duecker, and others, 2001) indicating some sort of upwelling forces at work.

Basaltic volcanism is another indicator of active, extensional tectonism. Late Cenozoic basalt flows abound in the state and Quaternary basalts are found in four places (Tweto, 1979). The Dotsero volcano erupted only 4,150 years ago (Giegengack, 1962).

Neogene faulting — Since Curtis (1975) first documented widespread, post-Laramide deformation in Colorado, the body of evidence continues to grow that active uplift and faulting is a dominant imprint on late Cenozoic geologic history. Late Cenozoic faults are common in the western two-thirds of Colorado (Figure 10). Steven (2002) concluded that major deformation took place in Colorado during latest Miocene and Pliocene time and continued into the Quaternary.

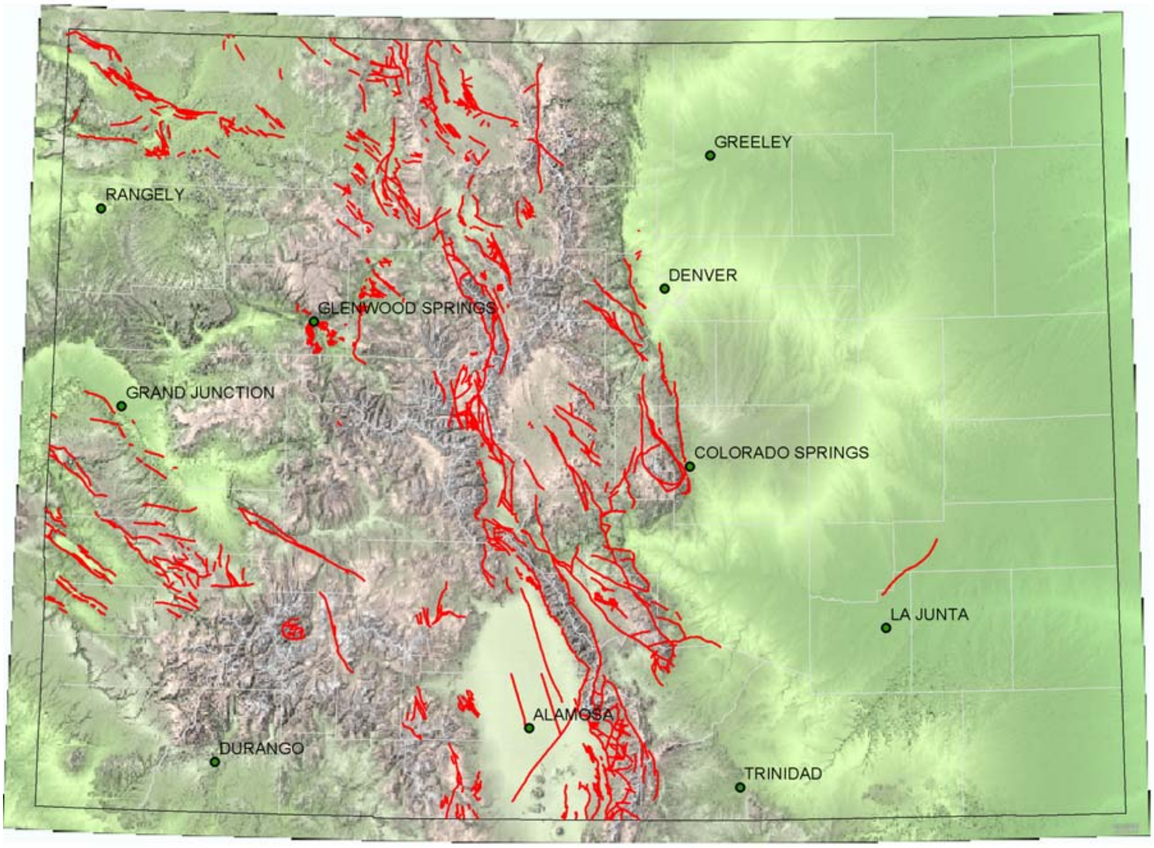


Figure 10. Known Late Cenozoic Faults in Colorado. These faults include Quaternary faults shown separately in Figure 1. The widespread Late Cenozoic faults (Miocene or younger) suggest that more Quaternary faults remain undetected. *from* Widmann and others (2002).

Apatite fission-track data from north-central Colorado demonstrate significant, post-Laramide uplift (Naeser, and others, 2002). They report that 4.0 km of material was removed from the Gore Range since middle Tertiary time.

Fault studies show large, vertical offsets of late Cenozoic rock units throughout central Colorado. For instance, geologic mapping in Rocky Mountain National Park (Braddock and Cole, 1990) shows two kilometers of post-Oligocene vertical displacement of volcanic rocks in Specimen Mountain; Geismann and others (1992) demonstrated 2.3 km of vertical offset of the ore body at

Red Mountain; coreholes at Climax verify 3.0 km of vertical displacement on the Mosquito fault (Wallace and others, 1968); Limbach (1975) reports 3.0 km vertical displacement of the Sawatch Range; and Lindsey, and others, (1986) report 4 km of vertical displacement of the Sangre de Cristo Range. These large faults span 150+ miles in central Colorado. With documented displacements of this magnitude and distribution, it is questionable whether one can safely make the assumption that a fault in Precambrian rock has not moved since the Laramide without strong evidence to that effect.

REASONS WHY COLORADO'S EARTHQUAKE INFORMATION IS LACKING

Colorado's database of information relative to earthquake hazard seems to be lacking for several reasons:

- A general perception among decision-makers that Colorado does not have an earthquake problem.
- Original uncertainty about the location of Colorado's 1882 Mw 6.6 earthquake.
- Difficulty in obtaining slip rates on Colorado's Holocene and Quaternary faults.
- Lack of statewide seismograph coverage making it difficult to accurately locate earthquakes and to detect smaller earthquakes.
- Short record of historic seismicity; approximately175 years versus 450 years in parts of the CEUS.
- Past research was not focused in Colorado because of higher priorities in other parts of the country.

CGS RESOURCES RELATIVE TO EARTHQUAKES AND FAULTING IN COLORADO

The Colorado Geological Survey has a number of resources that geotechnical practitioners might find useful in studying the earthquake hazard in Colorado:

OFR-03-4, "Published faults of the Colorado Front Range": Map plate and CD-rom contains faults published at a variety of scales (Morgan, 2003). This compilation vividly illustrates the incompleteness of our knowledge of the location and extent of faulting in the Front Range. It also is probably a good indicator of the lack of knowledge about faulting in other areas of the state.

Bulletin 52, "Colorado Earthquake Information, 1867-1996" CD-ROM (kirkham and Rogers, 2000). This publication received the 2001 "Excellence in the Use of New Technology Award" from the Western States Seismic Policy Council.

Earthquake Reference Collection: More than 500 papers on earthquakes and faulting relative to Colorado are available for review in the CGS offices. The collection includes many obscure studies and unpublished geotechnical reports. An online bibliography is at http://geosurvey.state.us.

Late Cenozoic Fault and Fold Database and Internet Map Server (Widmann and others (2002): This online publication (http://geosurvey.state.co.us/pubs/ceno/index.htm) is useful to quickly gain information about known faults that offset Late-Cenozoic (<23m.y.a) deposits in Colorado. Faults on the map server are color-coded by age of youngest known movement. Double clicking on a given fault brings up a data sheet containing a variety of information about the fault, e.g. length, sense of movement, geomorphic expression, age of faulted deposits, slip rate, and references.

SUMMARY AND CONCLUSIONS

Former Colorado State Geologist, Vicki Cowart, succinctly summed up our current state of knowledge about the earthquake hazard in Colorado, "We know enough, to *know*, that we *need* to know, a *lot* more."

The Colorado Earthquake Hazard Mitigation Council composed of seismologists; geologists; geotechnical, structural, and civil engineers; emergency managers; federal, state, and academic scientists; and insurance industry representatives issued the following consensus statement in 1999:

> "Based on the historical earthquake record and geologic studies in Colorado, an event of magnitude 6½ to 7¼ could occur somewhere in the state. Scientists are unable to accurately predict when the next major earthquake will occur in Colorado, only that one will occur. The major factor preventing the precise identification of the time or location of the next damaging earthquake is the limited knowledge of potentially active faults. Given Colorado's continuing active economic growth and the accompanying expansion of population and infrastructure, it is prudent to continue the study and analysis of earthquake hazards. Existing knowledge should be used to incorporate appropriate levels of seismic safety in building codes and practices. The continued and expanded use of seismic safety provisions in critical and vulnerable structures and in emergency planning statewide is also recommended. Concurrently, we should expand earthquake monitoring, geological and geophysical research, and mitigation planning."

ACKNOWLEDGEMENTS

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COLORADO FRONT RANGE SEISMICITY AND SEISMIC HAZARD

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Key Terms: earthquake, seismic, seismicity, crust, fault, hazard

ABSTRACT

Construction of seismic hazard and risk maps depends upon carefully constrained input parameters including background seismicity, seismic attenuation, and slip rates of Quaternary faulting. Incomplete knowledge of any of these parameters reduces the accuracy and usefulness of the resulting hazard maps. The state of Colorado has a rapidly growing population, active tectonics, and seismicity that has never been surveyed statewide. Monitoring of seismicity in Colorado has been sparse at best, and we have relied heavily on a patchy historical record and microearthquake surveys of limited spatial extent for our current state of knowledge regarding the levels of seismicity in Colorado. This paper summarizes two microearthquake experiments conducted in Boulder County and the Northern Front Range, Colorado, in 1996-7 and 1999, and describes the 1992 Rocky Mountain Front broadband seismic experiment. These brief experiments provide data that improves our understanding of Colorado seismicity and seismic hazard, though more comprehensive monitoring is needed. A brief discussion of mine blast practices is included, as mine blasts dominate the seismic record in Colorado at magnitudes less than 2. The potential for additional useful information on Colorado earthquakes and seismicity to come from additional analysis of existing seismic data sets is recognized.

INTRODUCTION

The level of seismicity in Colorado has been characterized as being low to moderate (Kirkham & Rogers 1981) due in part to the lack of adequate seismographic coverage in the state, and a number of sizable earthquakes have occurred in the historical and more recent record (Figure 1). The largest known historical earthquake in Colorado was the November 8, 1882 earthquake whose size (estimated Moment Magnitude 6.6 +/- 0.6 (Spence et al. 1996)) and location (somewhere in north-central Colorado) remain uncertain (McGuire et al. 1982; Kirkham & Rogers 1986; Spence et al. 1996). Perhaps the best known earthquakes in Colorado have been those induced by the disposal of waste fluids at the Rocky Mountain Arsenal near Denver (Evans, 1966; Healy et al. 1968; Herrmann, 1981) and secondary oil recovery in western Colorado at the Rangely oil field (Gibbs et al. 1973). Earthquake swarms in Colorado are not uncommon (Bott & Wong 1995). A swarm of earthquakes, including one of magnitude 4.6, occurred near Trinidad, Colorado in the fall of 2001 (Meremonte et al. 2002). The largest instrumentally recorded natural earthquake in Colorado was a magnitude 5.5 earthquake in 1960 which occurred near Ridgeway in southwest Colorado (Talley & Cloud 1962).

As noted above, earthquakes have occurred in geographic locations spread throughout the region. Occasional microearthquake surveys of limited extent have been conducted (e.g. Goter & Presgrave 1986; Keller & Adams 1975; Bott & Wong 1995; Sheehan, 2000; Godchaux, 2000) and the U.S. Bureau of Reclamation has operated two small seismograph networks in southwest Colorado, mainly to monitor induced seismicity related to a dam near Ridgeway, Colorado, and deep brine injection near Paradox, Colorado. In addition, Microgeophysics Corporation operated a seismograph network in the Front Range under contract to the Denver Water Board from 1983-1993. Current seismic station coverage in the state of Colorado is extremely poor, with only stations at Idaho Springs, Great Sand Dunes, and Paradox Valley (southwest Colorado) reporting times to the NEIC. Thus, the PDE catalog offers an extremely incomplete view of Colorado seismicity, with a magnitude threshold of about 3.0 and large location uncertainties in most parts of the state (Matthews, 2002).

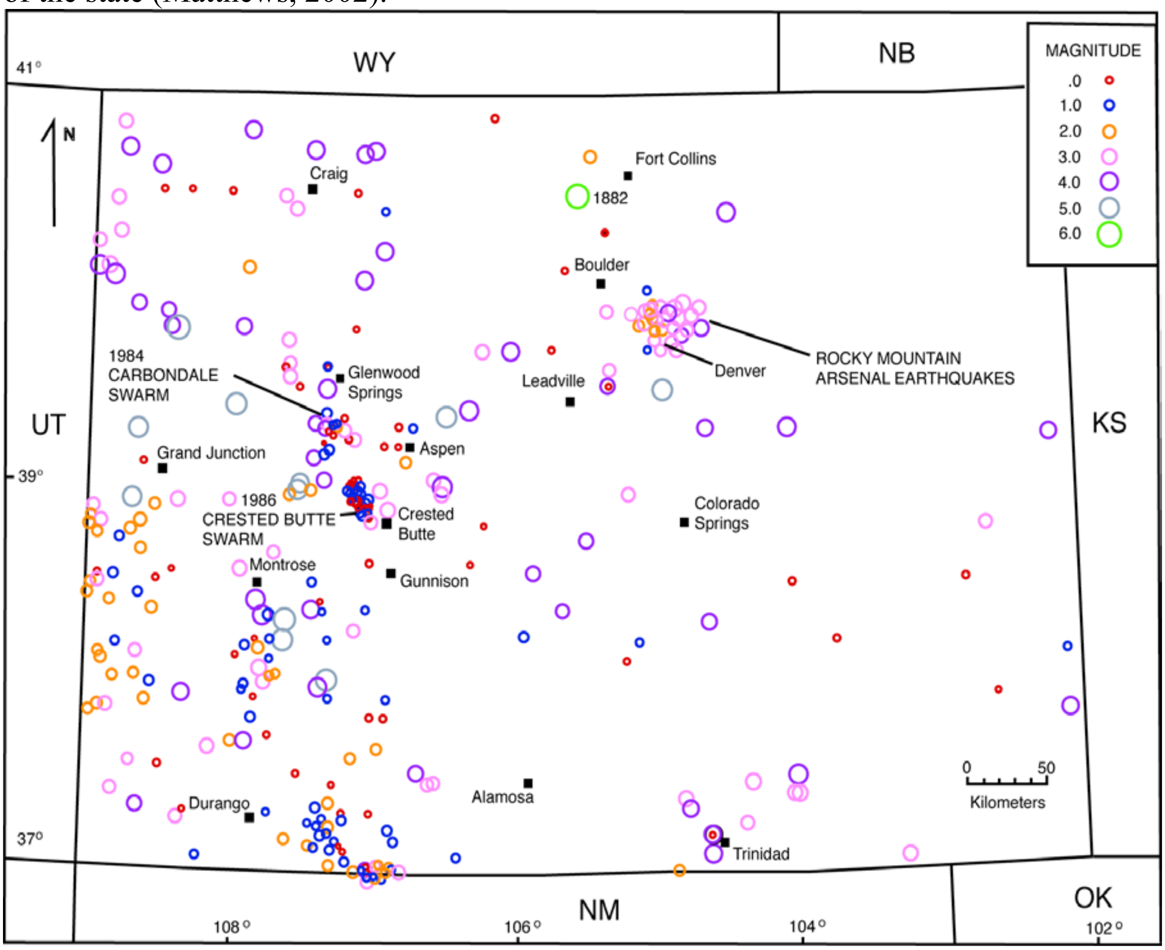

Figure 1. Seismicity of Colorado and surrounding areas, 1870-1992 (after Bott & Wong1995). Dates of significant earthquakes are included along with observed swarms. Earthquakes denoted by circles, color and size corresponds to magnitude. Cities shown as black squares.

Despite the low to moderate levels of historical seismicity, there is significant geologic evidence of late Quaternary tectonic activity throughout the state of Colorado (Figure 2). Widmann et al. (1998) catalog 92 Quaternary faults around the state. The scarp-morphology data from the Rio Grande rift in south-central Colorado suggest that the youngest movements on the scarps occurred between 5000 and 15,000 years ago (Colman 1986). On the Sangre de Cristo Fault Zone in south-central Colorado, within the northern Rio Grande Rift Zone, Quaternary deposits show multiple events with individual displacements of 1.7 to 2.9 m, and fault scarp data suggest past earthquakes of M_L 7.0-7.3 (McCalpin 1986). Fourteen faults (Figure 2) around the state have been assigned Maximum Credible Earthquakes from M 6.25 – 7.5 (Widmann et al. 1998).

The contemporary crustal stress regime in Colorado includes extension along a roughly northeast-oriented axis (Bott & Wong 1995). A possible mechanism for seismogenesis in the area is reactivation of existing faults, which are oriented favorably to the contemporary stress field, i.e. oriented west-northwest to northwest. Focal mechanisms of two of the earthquakes related to the Rocky Mountain Arsenal fluid injection have indicated normal (or extensional) fault movement locally in the northern Front Range region (Wong 1986).

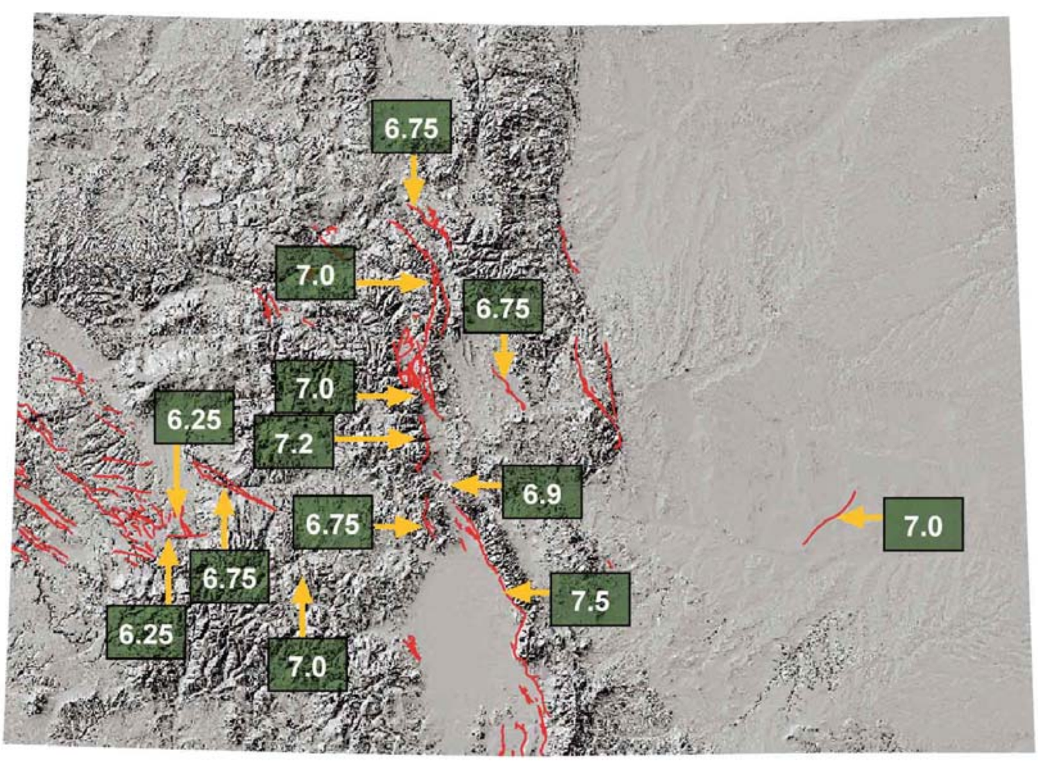

Figure 2. Maximum credible earthquakes in Colorado. The Quaternary faults shown on this map have been studied and assigned a maximum credible earthquake based on the length of the fault, the age of the latest movement, and the recurrence interval for past earthquakes (Widmann et al. 1998).

BOULDER MICROEARTHQUAKE SURVEY

In the fall of 1996, a small seismic array was deployed around Boulder, Colorado, to map local seismicity patterns and to provide hands-on training in earthquake seismology to undergraduate geology students at the University of Colorado. Using three sensors deployed by the field geophysics class, and incorporating data from two other local United States Geological Survey (USGS) stations, earthquakes within a radius of approximately 50 km of the array were recorded and analyzed. Results and pedagogy from this project are summarized in Sheehan (1999).

Instrumentation and Deployment

The seismic instrumentation used in the Boulder earthquake deployment consisted of three Reftek 72A-08 data acquisition systems and three Mark Products L4-3D three-component seismometers. The instruments were deployed at Sugarloaf Mountain (SUG), Louisville (LVL) and Table Mountain (TBL), with station spacing of approximately 15 km (Figure 3). The instruments were in place from August of 1996 through November of 1996, with one station (TBL) left in place through September 1997. Each station recorded continuously at 40 samples/s with an additional triggered data stream of 100 samples/s. USGS stations used to complement the Boulder network included stations ISCO (Idaho Springs, Colorado) and GLD (Golden, Colorado).

Data Analysis and Results

The program HYPOINVERSE (Klein 1978) was used to determine locations of earthquakes recorded by the Boulder microearthquake network. A crustal velocity model for the Colorado Front Range from Prodehl and Lipman (1989) was used. Magnitudes were determined for each event using a coda duration method (e.g., Lawson 1978). For each event the magnitude was determined using the following formula:

$$M_{dur} = 1.86[\log(\text{coda length})] - 0.85 \qquad (1)$$

where coda length is given in seconds. Our duration magnitude scale was determined by regression of three events recorded by both the Boulder network and the USGS NEIC.

Events located during this experiment are shown on Figure 3. Twenty-three small seismic events were located within a 50-mile radius of Boulder during the first 3 months of the deployment, with an additional 24 events located during 1997. The earthquakes ranged from duration magnitude 1.0 to 2.9, and include a cluster of five magnitude 2.6 - 2.8 earthquakes located near Castle Rock, Colorado. An alignment of microearthquakes along a northwest-southeast trending line just west of Golden, Colorado agrees with the geometry of a previously recorded alignment of microearthquakes (Unruh et al. 1994) and correlates well with generally northwest-southeast trending fault geometries of the area (Bott & Wong 1995; Wong 1986; Widmann et al. 1998). The majority of other events were located almost exclusively in the mountains and foothills west of Denver rather than in the plains to the east. No events were found near the former Rocky Mountain Arsenal northeast of Denver, which was the site of extensive induced seismicity in the late 1960's and early 1970's (Evans 1966; Healy et al. 1968; Herrmann 1981). The map also shows a clustering of events near a known hard rock quarry in Clear Creek Canyon 20 km southeast of station ISCO (Idaho Springs, Colorado) and another approximately 20 km southwest of station ISCO. Events were identified as blasts based on location, origin time statistics, and waveform character. Blasts were excluded from the catalog and the b-value statistics. Thirty-one events from the sparse one year deployment were classified as earthquakes, with many more events identified as quarry blasts.

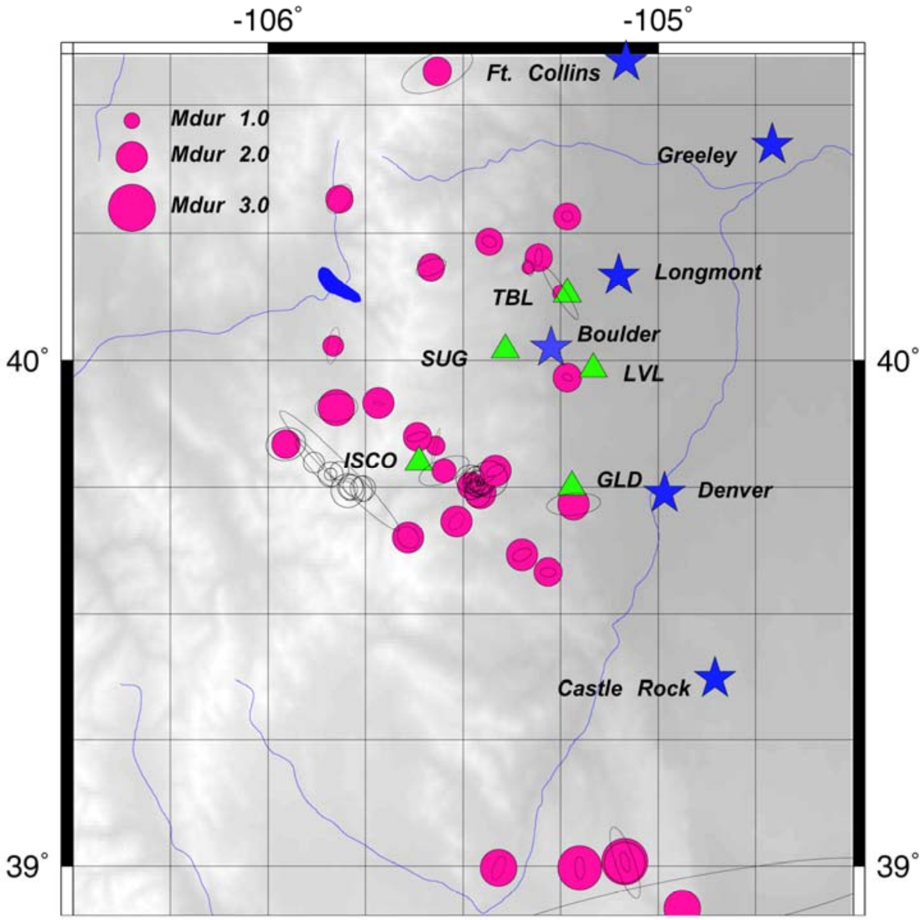

Figure 3. Seismic stations (green triangles), seismic event locations (red circles), and cities (blue stars) on grey-scale topographic map. Earthquakes are shown as filled circles, suspected quarry blasts are shown as open circles. Many blasts were removed from the catalog and are not shown. Size of circles correspond to earthquake magnitude. Formal earthquake location errors are denoted by ellipses centered on each event

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An earthquake frequency analysis was performed and compared to sample catalogs from California and western Nevada (Gross & Jaume 1995) and New England (Doll & Toksoz, 1997) (Figure 4).

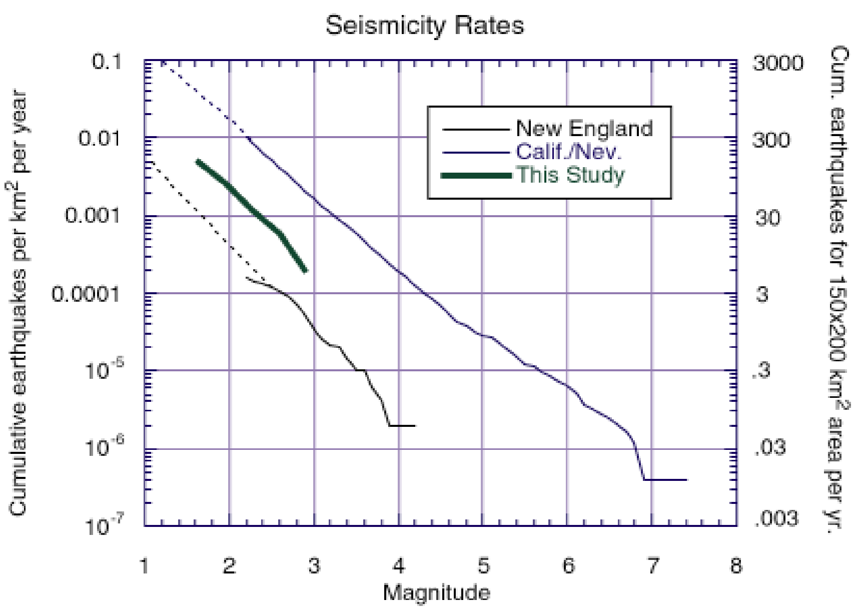

Figure 4. Seismicity from the Boulder microearthquake experiment compared to seismicity catalogs from New England and California, normalized to events per square km per year (left axis) and rescaled to area the size of the northern Front Range (right axis).

Both the California and New England catalogs represented a five-year period. All data were normalized to earthquakes per square kilometer per year. The b-value plot for this small sample suggests a seismicity level for the Front Range approximately midway between that of the California/Nevada region and that of New England (Figure 4). The Colorado values obtained from this deployment are comparable to seismicity levels estimated from historic records of earthquakes in Colorado (Unruh et al 1994). An extrapolation of the line for this region suggests a recurrence interval for a magnitude 6 earthquake to be between 100 and 1000 years for the Colorado Front Range (approximately 200 km x 150 km area). However, the window of magnitudes sampled by this project is narrow and this extrapolation is tenuous. A permanent or long-term array in the area would produce more reliable recurrence intervals.

CONTINENTAL DYNAMICS OF THE ROCKY MOUNTAINS (CD-ROM) EXPERIMENT

A microearthquake survey was performed in north-central Colorado during the summer of 1999. The deployment was part of the Continental Dynamics of the Rocky Mountains (CD-ROM) project (Karlstrom et al. 2002). The passive source CD-ROM deployment included 25 broadband seismometers deployed in a NW-SE line between Steamboat Springs, Colorado, and Rawlins, Wyoming from June 1999 through August 2000. The main objectives of the CD-ROM passive source (earthquake) deployment was to study crust and mantle structure beneath the stations using distant earthquakes as sources. The linear geometry of the array was designed to best image the deep crust and mantle beneath Proterozoic suture zones in the region. A line of seismograph stations is not a good geometry for studies of local earthquakes, which are better located when stations surround the earthquake. In the summer of 1999, six short period seismometers were deployed in an arc 50 to 150 km east of the CD-ROM line, to supplement the main CD-ROM line and improve our ability to locate earthquakes within the northern Front Range region (Figure 5). The array surrounds the probable source region of the 1882 M6.6 northern Colorado earthquake, and crosses many ancient crustal suture zones and regions of Laramide faulting and deformation (Karlstrom et al. 2002). The six short period stations were deployed surrounding the northern Front Range near the following towns: Silverthorne, Boulder, Estes Park, Poudre Park, and Virginia Dale, Colorado, and Laramie, Wyoming.

Seismic instrumentation included Mark Products L22 2-Hz seismometers for the short period stations, and a mix of Streckeisen STS2, Guralp CMG40T, and Guralp CMG3T broadband seismometers for the CD-ROM broadband array. All data was recorded in the field using Reftek Data Acquisition Systems (DAS). Data was recorded at a sample rate of 25 sps for the summer of 1999, with a lower sample rate during the winter months. The short period stations were powered by 110 V AC electrical outlet, using an AC/DC converter between the AC outlet and the DAS, and the broadband stations were powered by car batteries connected to solar panels. The use of AC power for the short period stations restricted the possible deployment sites considerably and resulted in some noisy stations. All data were recorded to disk at the station, and data was retrieved by periodic station service visits.

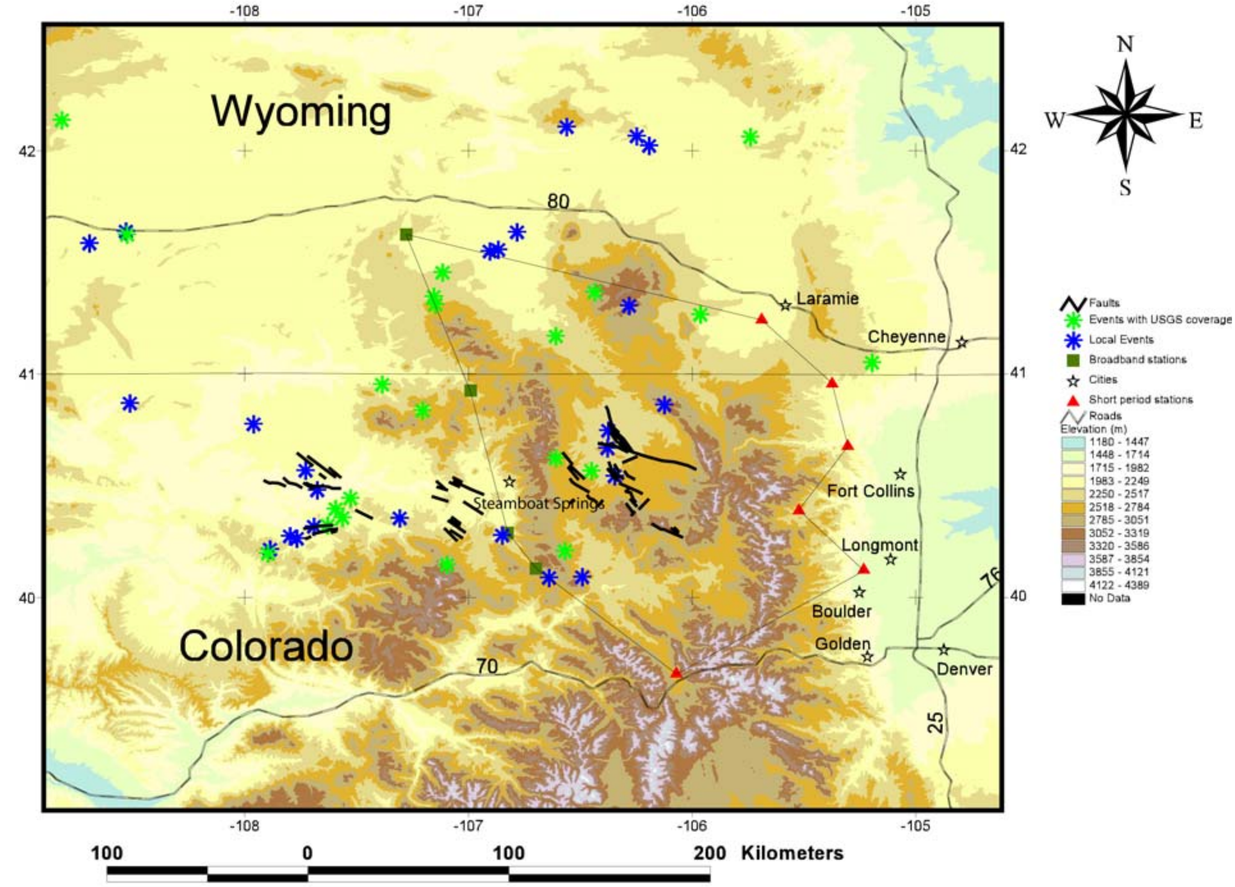

Figure 5. Continental Dynamics of the Rocky Mountains (CD-ROM) seismograph stations (black squares) and supplementary short period stations (red triangles) . Black lines are roads, locations of cities given by stars. Faults from Tweto (1979) designated by bold black lines. Asterisks denote seismic event locations, blue are unique to this study, green also appeared in USGS National Earthquake Information Center (NEIC) catalog and were used to calibrate our magnitude scale.

The program HypoInverse (Klein 1978) was used to determine earthquake hypocenters. Phase arrival time picks were made mainly with the computer program pql (Passcal quick look) and transferred to HypoInverse input files. The velocity model used was from Wong (1991) and consists of a three-layer crust over a half space. Magnitudes were determined using a duration magnitude scale. The scale was empirically derived on a station by station basis through regression of the duration of eleven events recorded by this deployment and the magnitude of those same events reported by the USGS NEIC.

Forty-seven local events with magnitude from Mdur 1.1 to Mdur 3.4 were recorded during thirty-four days of continuous seismic monitoring of the six short period stations and selected CDROM broadband stations. Sixteen of these seismic events were located within the array. Earthquake locations and mapped faults are shown in Figure 5. A cluster of five events is found east of Steamboat Springs, and is close to previously mapped faults (Tweto 1979). A line of seismic events trending N60E was found 50 km west of the CD-ROM line. This trend is coincident with the fabric of Precambrian basement fractures in the region. Neither of these trends were on the modern faults summarized by Widmann et al. (1998). Consequently, it is not yet recognized that either of these two areas possess any active fault movements. The N60E trend of earthquakes is parallel to the extension direction estimated for the region by Zoback & Zoback (1989). Time of day statistics suggest that perhaps a quarter of the events located are actually mine blasts, and are removed from our catalog. More work remains to be done to fully utilize the complete one year of data recorded by the CDROM experiment.

MINING RELATED SEISMICITY

We must take care to distinguish between man-made blasts and earthquakes, with primary initial discrimination criteria being location and timing. Our depth resolution is insufficient to use as a discriminant. Waveform character is also used, as similar blast waveforms become recognizable to analysts with experience, as we have found in our own microearthquake deployments described previously (Sheehan 2000; Godchaux 2000). Possible contamination of seismic catalogs with blasts is a common problem that all local network operators must deal with to extend catalogs to low magnitude range (Agnew et al. 1990; Dewey 1998; Rydelek & Hass 1994; Weimer & Baer 2000). Most mine blast activity in Colorado occurs in the afternoon (Dewey, 1998). Some of the mining events cataloged by the USGS NEIC from Colorado are planned longwall collapses in underground mines, and these occur around the clock. Figure 6 shows times of surface-mine explosions and longwall-mine collapses for western Colorado.

Our work focuses on events with $M > 2$, as it can still be difficult to confidently separate out random construction and hard rock mining blasts at magnitudes less than 2. Additional information on mining induced seismicity can be found at http://neic.usgs.gov/neis/mineblast/evidence.html. With both the Northern Colorado Front Range Array and the Boulder county network, a peak in seismicity is observed at 2200 UTC (late afternoon). These events are typically excluded from further analysis, in an attempt to reduce the number of mine blasts in the catalog.

ROCKY MOUNTAIN FRONT PASSCAL EXPERIMENT

In May through December of 1992, a large NSF-funded Program for Array Studies of the Continental Lithosphere (PASSCAL) experiment was conducted across the Rocky Mountain Front and extending into the Great Plains and Colorado Plateau (Lerner-Lam et al. 1998). The experiment included the deployment of thirty three-component broadband seismometers at a nominal station spacing of 75 km throughout the state of Colorado (Figure 7). Although the main purpose of the deployment was to record distant earthquakes (teleseisms) in order to image the crust and upper mantle under the Rocky Mountain Front, the continuously recording instruments

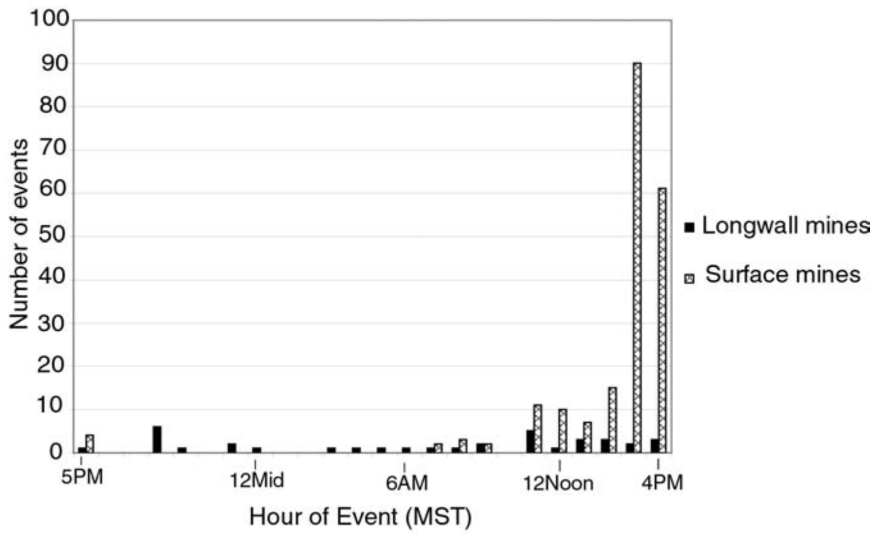

Figure 6. Time of day of longwall mining collapses and surface mine blasts (Figure from J. Dewey).

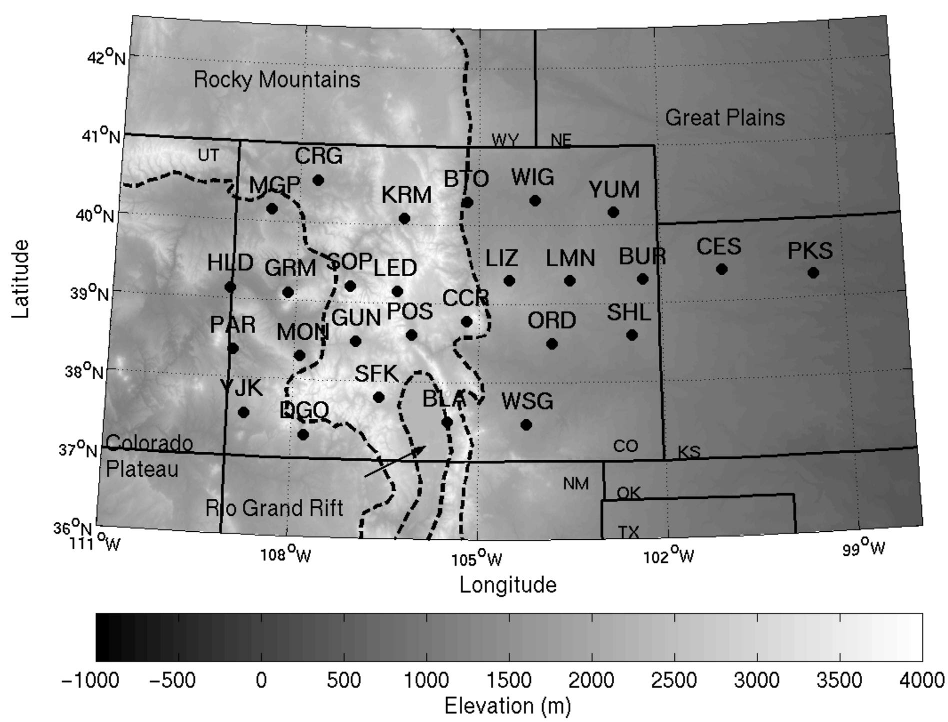

Figure 7. Topographic map of the Western United States, with PASSCAL portable broadband seismic stations (black circles) deployed in 1992 Rocky Mountain Front PASSCAL Experiment (Lerner-Lam et al., 1998).

recorded local events as well as teleseisms. Data was collected from each station in both continuous (10 samples per second) and triggered (20 samples per second) data streams. A total of over 50 Gb of continuously recorded data was collected. The teleseismic events were parsed from the full dataset and sent to the IRIS Data Management Center in Seattle in 1994. The continuous data and local events have not been archived and are on aging field tapes. We have performed feasibility studies to ensure that we can parse out local seismograms of good quality (Figures 8-9). In addition to seismicity studies, the Rocky Mountain Front data can be used for crustal attenuation and site effects studies that will also be of great utility for seismic hazard analysis.

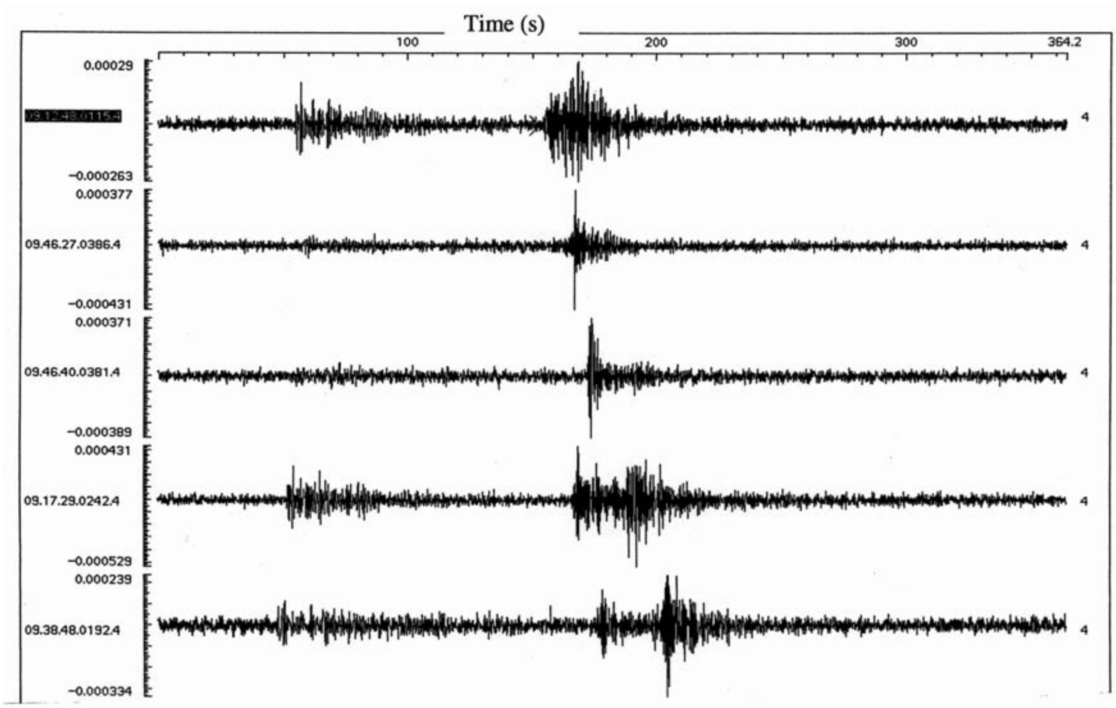

Figure 8. Sample vertical component seismograms for earthquake of June 2, 1992, 09:54:09.51 UTC, 38˚ 0.36' N, 107˚ 40.49' W, USBR Network magnitude 2.4. Seismograms shown are (from top to bottom) from stations MON, GUN, SOP, PAR, HLD of Rocky Mountain Front array and are ordered by P wave arrival time (see Figure 8 for station locations). Waveforms have been bandpassed from 1-5 Hz, and amplitudes are scaled by trace. P arrival at top station marked by an X on the waveform at approximately 150 s.

Seismicity

The limited deployment of the Rocky Mountain Front seismic array (six months) gives us only a snapshot of seismic activity in the state. However, studies to date of Colorado seismicity and recurrence intervals have relied on extremely incomplete data sets because that is all that is available (Unruh et al. 1994; Charlie et al. 2002). Analysis of the Rocky Mountain Front PASSCAL data set will contribute significantly to the existing body of knowledge regarding

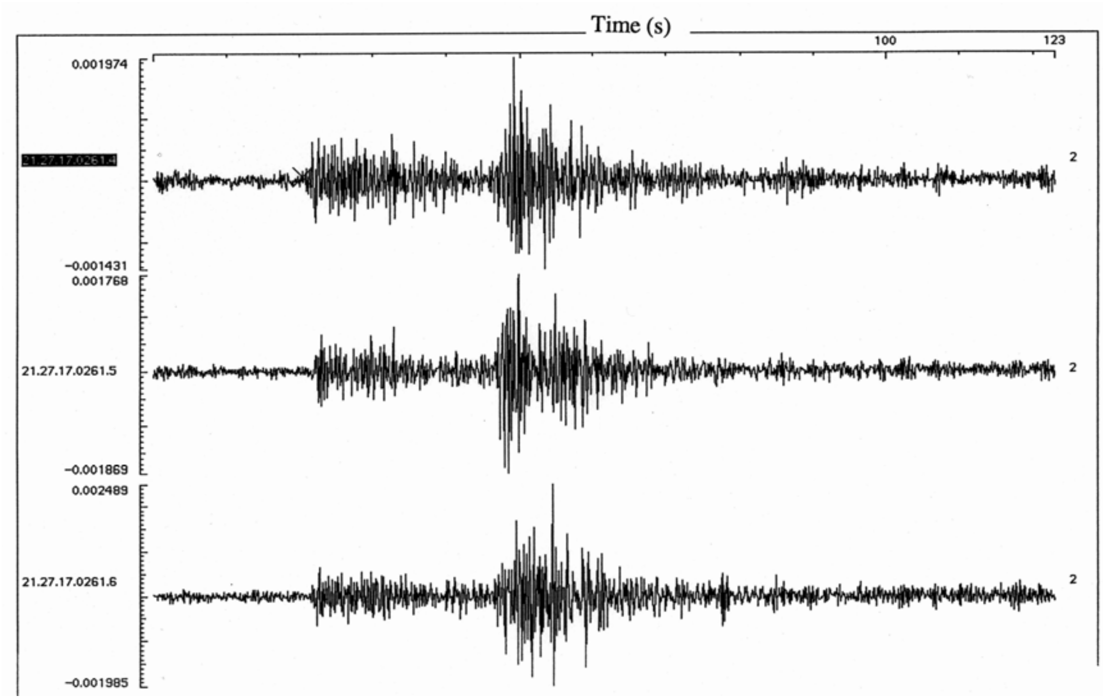

Figure 9. Vertical, north-south, and east-west seismograms recorded at Rocky Mountain Front station SFK for earthquake of October 20, 1992, 21:35:41.44 UTC, 38° 46.83' N, 108° 31.76' W, USBR Network magnitude 3.0. Waveforms have been bandpassed from 1-5 Hz, and are scaled by the maximum amplitude in each trace.

location, frequency, and size of earthquakes in the state of Colorado. Since the state of Colorado has never been blanketed with broadband three-component seismometers in this manner, the contribution to our understanding of the nature and orientation of tectonic stress regimes and associated seismic risk within the state of Colorado is likely to be substantial. The improved spatial coverage will allow for improved epicenter location and accurate depth determination throughout the entire state of Colorado for the first time ever; it also provides a much more uniform distribution of stations on a focal sphere for earthquakes in the state. The proposed Earthscope/USArray project, if funded, will also cover the state with a similar density of seismometers for a period of two years (Levander et al. 1999). The USArray dataset will be a valuable resource for studying Colorado seismicity.

We estimate that the detection/location threshold of the Rocky Mountain Front array will be comparable to that of microearthquake networks of similar dimensions, on the order of magnitude 1.5 for events within the array (Sheehan & Steeples, 1983; Harvey, 1994; Bratt & Bache, 1988). It is possible that our low sampling rate (10 to 20 samples per second) could increase the threshold for accurate locations to the magnitude 2.0 range. We have obtained catalogs for the June to December 1992 time period from the Ridgeway and Paradox Valley networks in southwest Colorado operated by the United States Bureau of Reclamation, and the Front Range network operated by Microgeophysics Corporation for the Denver Water Board (Figure 11). A total of nearly 150 earthquakes were recorded and located in this six month period by these arrays. The events ranged from negative magnitudes to magnitude 3.1, with the majority (65) in the magnitude 1.0 to 2.0 range, and a significant number (35) with magnitude greater than 2.0. None of these events were reported in the USGS PDE's (Preliminary Determination of Epicenters catalog). We have examined a sample of the Rocky Mountain Front data for some of these events, and have found that they were well recorded by the Rocky Mountain Front array, even for events in the magnitude 1.0 range (Figures 9-10). It is important to note that these arrays have limited spatial coverage (cover less than 10% of the state) whereas the Rocky Mountain Front array covered the entire state uniformly. Not surprisingly, we have identified a number of events (Figures 11-12), that are well recorded by the Rocky Mountain Front array that do not appear in any other catalogs, including the PDE, Utah Network, Ridgeway and Paradox Valley Networks, and Front Range Network. To date we have scanned only a small portion (about 2%) of the data for local events, and have found 22 probable events that did not appear in any other earthquake catalogs. Assuming that this random sample scales linearly, we might expect to find 1000 new events to add to the Colorado earthquake catalog. If even as low as 10% of these events are locatable and are classified as earthquakes, we would still have over 100 earthquakes from portions of Colorado that have been virtually unstudied.

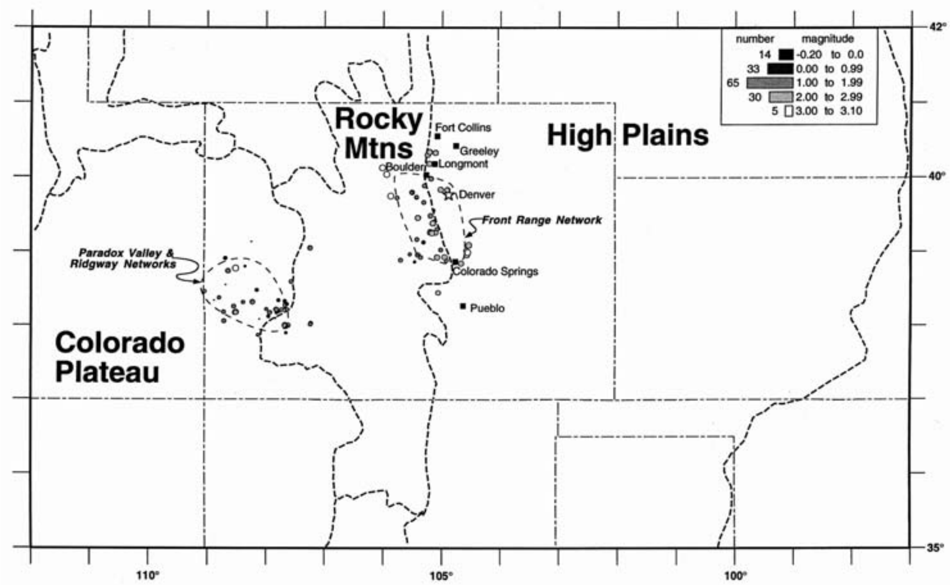

Figure 10. Colorado seismicity for the period June through December 1992 for Front Range and Paradox Valley/Ridgeway Networks. Thin dashed lines indicate approximate spatial extent of the networks. This figure illustrates the background levels of seismicity that are revealed whenever seismograph stations are deployed. There are likely similar levels of seismicity throughout the state of Colorado, but there is insufficient seismic station coverage to detect and locate these small earthquakes.

Earthquake Source Parameters

Determination of earthquake source parameters, primarily earthquake focal mechanisms, seismic moment, and in some cases stress drop, for well-recorded earthquakes is an important component of earthquake hazard analysis. Since 1992 there have been 16 earthquakes of magnitude > 3.5 within Colorado reported by the NEIC. Examples include an M 4.5 earthquake near the town of Ridgway in southwest Colorado on September 13, 1994, an M 4.1 earthquake on Christmas Day 1994 which occurred 30 km south of Denver, an M 4.1 earthquake in northwestern Colorado on March 20, 1995, and M 4.6 Trinidad sequence of August/September 2001 (Meremonte 2002).

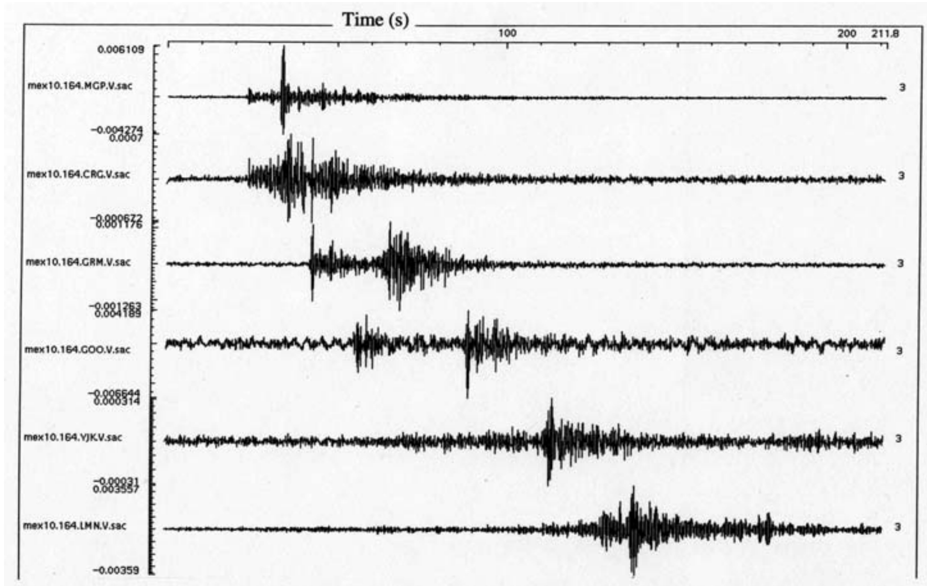

Figure 11. Vertical components of seismograms for event of July 27, 1992, 20:05:45 UTC, as recorded on stations CRG, MGP, GRM, HLD, BTO, PAR (see Figure 7 for station locations). Stations ordered by P wave arrival time. Closest station is CRG, near town of Craig in northwest Colorado. Waveforms have been bandpassed from 1-5 Hz. This event produced high quality three-component waveforms on the Rocky Mountain Front stations, but did not appear in the PDE, Utah Network, or U.S. Bureau of Reclamation catalogs (as either an earthquake or a blast).

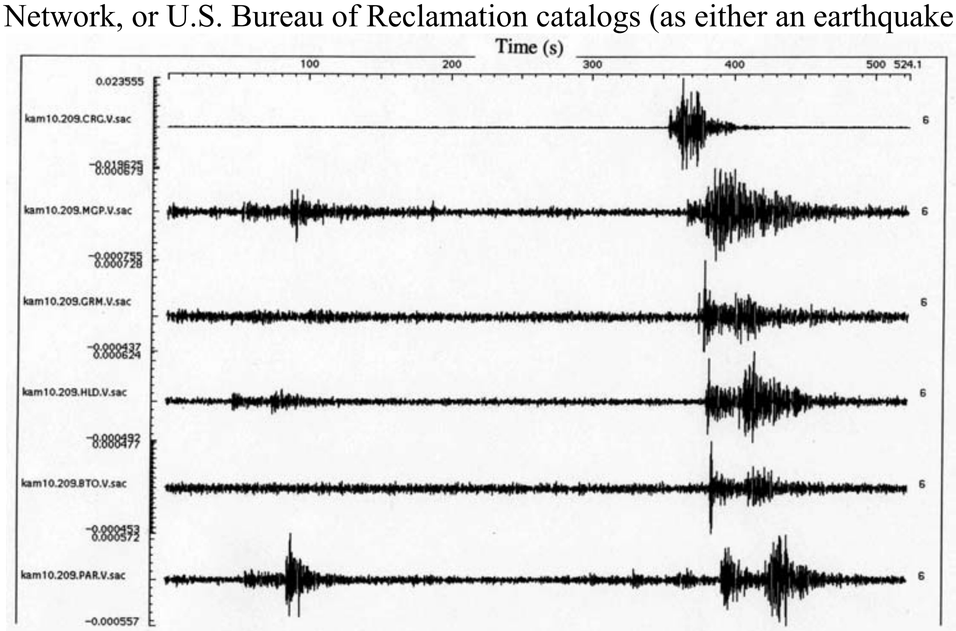

Figure 12. Vertical components for seismic event of June 12, 1992, 01:00:50 UTC as recorded on stations MGP, CRG, GRM, GOO, YJK, LMN. Stations ordered by P wave arrival time. Closest station is MGP, in northwest Colorado (see Figure 8 for station locations). Waveforms have been bandpassed from 1-5 Hz. This event produced high quality three-component waveforms on the Rocky Mountain Front stations, but did not appear in the PDE, Utah Network, or U.S. Bureau of Reclamation catalogs. A smaller event is visible at approximately 50 s. Figures 11 and 12 illustrate earthquakes or blasts that occur in Colorado but because of inadequate station coverage go unreported.

At this time there are less than twenty published focal mechanisms for Colorado earthquakes. Additional focal mechanism work may show where the state of stress changes from the extensional regime of the Rio Grande Rift to the compressional regime of the Eastern U.S. It is important to know which regime the Front Range Urban Corridor is in. This knowledge of the state of stress can provide the basis for assessing the current style and orientation of active faults and for determining their potential for producing earthquakes. It is also important to have an idea what typical stress drops are in the region, which will affect the moment release associated with a given size fault. Intraplate (plate interior) earthquakes tend to have higher stress drops than interplate (plate boundary) earthquakes, and it is unclear which regime the Rocky Mountains fall under. Earthquake source parameters can be determined through the use of full waveform inversion (e.g. Fan & Wallace 1991; Walter 1993; Randall et al. 1995; Ammon et al. 1995). Full waveform inversion techniques offer several advantages over first motion analyses for earthquakes from a regional network, particularly improved depth control, focal mechanisms, and seismic moment. Results of earthquake focal mechanism studies within western and central Colorado indicate a regime of northeast-oriented extensional stress (Wong, 1986; Wong & Humphrey 1989) in contrast to the compressional regime suggested for western Colorado by Zoback & Zoback (1980). Analysis of more earthquake focal mechanisms will provide valuable information needed to update and revise the Zoback & Zoback (1980, 1989) stress province boundaries within Colorado. Earthquake focal mechanisms can and should be determined for all earthquakes greater than magnitude 4.0 in Colorado since 1992. The Rocky Mountain Front experiment stations can be used for the 1992 earthquakes, and the US National Seismic Network Stations can be used for post-1992 earthquakes

SUMMARY AND CONCLUSION

Seismicity studies as described here allow us to determine the occurrence, distribution, and source properties of earthquakes and relate seismicity to geologic structures and tectonic processes throughout the rapidly growing state of Colorado. This work aids in characterization of earthquake potential in the southern Rocky Mountains and eastern Colorado Plateau regions and in the identification of active faults and seismic risk throughout Colorado. The current seismic station coverage in Colorado is inadequate to detect most earthquakes less than magnitude 3.5, and the location errors on detected earthquakes are large. Temporary seismic network deployments give some indication of the background levels of seismicity in the state, but more uniform and longer term seismic station coverage is needed to give a representative sample of earthquake potential in the state. Further analysis of existing data sets could shed additional valuable insight into Colorado seismicity.

ACKNOWLEDGEMENTS

We thank landowners who allowed us to install seismograph stations on their property. Jim Dewey provided Figure 6 and shared information on mine blast practices and active mine locations in Colorado. Glenn Kroeger assisted with the processing of the local earthquake data from the CD-ROM experiment. The author benefited from discussions with Vince Matthews and thanks him for the use of Figure 2. Vince Matthews and Pat Rogers provided helpful reviews, and editor Doug Boyer provided much assistance.

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Appendix 1

Hypoinverse locations of picked events with calculated magnitudes for CD-ROM microearthquake experiment. Magnitude 1 is either a calculated USGS magnitude (M_L) or a calculated duration magnitude (M_{dur}) in which the coda length runs from the P-wave arrival to the point at which the event can no longer be distinguished from the background noise. Magnitude 2 is calculated duration magnitude in which the coda length begins with the p-wave arrival and ends when the signal falls to twice the level of the background noise. It is likely that construction blasts and quarry blasts remain in the catalog and this list should be used with caution.

Date (Day-Month)	Origin Time (hh.mm.ss.ss; UTC)	Latitude	Longitude	Depth (km)	Magnitude 1	Magnitude 2
22-Jun	16.07.28.13	40.5667	-107.7283	8.12	2.2 M_{dur}	2.3 M_{dur}
23-Jun	19.08.59.96	41.3655	-106.4367	0.88	3.1 M_{dur}	3.0 M_{dur}
23-Jun	22.28.29.78	40.7742	-107.9612	7.05	2.0 M_{dur}	2.0 M_{dur}
24-Jun	20.46.39.97	40.3547	-107.3083	4.82	2.5 M_{dur}	2.5 M_{dur}
24-Jun	21.13.27.07	40.5413	-106.3460	1.10	2.6 M_{dur}	2.7 M_{dur}
25-Jun	06.35.14.41	42.0645	-106.2488	0.01	2.6 M_{dur}	2.1 M_{dur}
25-Jun	22.05.19.05	40.2775	-107.8503	0.05	3.0 M_{dur}	2.8 M_{dur}
26-Jun	23.56.44.61	40.2067	-106.5693	7.01	2.8 M_{dur}	2.8 M_{dur}
27-Jun	19.52.33.90	41.6233	-108.5263	0.11	2.8 M_{dur}	2.9 M_{dur}
28-Jun	10.25.11.37	41.6333	-106.7837	7.37	2.0 M_{dur}	1.7 M_{dur}
28-Jun	18.17.05.42	41.5840	-108.6962	0.03	3.0 M_{dur}	2.6 M_{dur}
29-Jun	12.52.24.28	41.3105	-107.1493	7.00	2.6 M_{dur}	2.8 M_{dur}
29-Jun	19.34.43.10	42.0600	-105.7395	0.11	2.7 M_{dur}	2.7 M_{dur}
1-Jul	22.05.10.85	40.7467	-106.3758	3.22	2.8 M_{dur}	2.7 M_{dur}
1-Jul	22.20.43.12	40.6228	-106.6100	10.37	2.6 M_{dur}	2.7 M_{dur}
2-Jul	11.49.14.46	42.0200	-106.1922	16.57		
2-Jul	15.46.54.28	41.6385	-108.5320	0.03	2.2 M_{dur}	2.0 M_{dur}
2-Jul	18.15.53.00	40.9528	-107.3863	9.05	2.0 M_{dur}	2.1 M_{dur}
3-Jul	05.13.06.07	42.1053	-106.5613	26.75		
3-Jul	05.41.59.59	41.3048	-106.2832	34.53		
4-Jul	21.57.27.75	40.2152	-107.8888	0.03	3.0 M_{dur}	2.9 M_{dur}
4-Jul	14.37.52.25	41.3455	-107.1558	0.11	2.9 M_{dur}	3.1 M_{dur}
5-Jul	22.09.37.87	40.3213	-107.6307	9.37	3.2 M_{dur}	3.0 M_{dur}
6-Jul	22.01.52.28	40.1973	-107.8968	0.20	1.7 M_{dur}	1.8 M_{dur}
6-Jul	22.06.24.26	40.1473	-107.0973	15.24	3.2 M_{dur}	3.2 M_{dur}
7-Jul	15.15.32.67	40.5655	-106.4513	8.56		
7-Jul	20.19.29.45	40.8365	-107.2052	0.11	2.7 M_{dur}	2.5 M_{dur}
7-Jul	22.03.00.99	40.3167	-107.6907	7.00	1.6 M_{dur}	1.4 M_{dur}
8-Jul	19.12.53.42	45.4233	-105.6243	7.00	3.4 M_{dur}	3.4 M_{dur}
8-Jul	20.24.13.30	40.4790	-107.6765	7.35	2.7 M_{dur}	2.7 M_{dur}
8-Jul	21.58.07.03	40.2637	-107.7685	6.90	2.7 M_{dur}	2.7 M_{dur}
11-Jul	23.11.35.30	41.0507	-105.1957	7.00	2.8 M_{dur}	2.7 M_{dur}

Date (Day-Month)	Origin Time (hh.mm.ss.ss; UTC)	Latitude	Longitude	Depth (km)	Magnitude 1	Magnitude 2
12-Jul	22.00.24.24	40.2740	-107.7983	0.03	2.6 M_{dur}	2.8 M_{dur}
15-Jul	19.26.30.13	40.4442	-107.5277	0.27	2.9 M_{dur}	2.9 M_{dur}
16-Jul	21.35.10.82	40.6667	-106.3792	5.48	1.3 M_{dur}	1.1 M_{dur}
18-Jul	15.12.55.27	41.2658	-105.9643	15.62	3.3 M_{dur}	3.3 M_{dur}
18-Jul	22.25.28.45	40.3557	-107.5647	2.41	2.6 M_{dur}	2.6 M_{dur}
19-Jul	04.17.51.63	42.1347	-108.8183	7.00	1.6 M_{dur}	1.5 M_{dur}
19-Jul	10.27.09.75	40.8680	-108.5152	0.05	2.9 M_{dur}	2.9 M_{dur}
20-Jul	20.15.02.76	41.1682	-106.6095	0.11		
21-Jul	02.36.27.49	41.4525	-107.1168	13.21	2.8 M_L	

QUANTIFYING SEISMIC HAZARD IN THE SOUTHERN ROCKY MOUNTAINS THROUGH GPS MEASUREMENTS OF CRUSTAL DEFORMATION

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Key Terms: seismicity, crustal deformation, GPS, Rocky Mountains, hazard

ABSTRACT

Using repeated high precision GPS measurements on existing monuments, we hope to accurately determine the present day crustal strain rates in the State of Colorado on a regional and/or local scale. We will provide the first modern, space-based crustal strain and surface velocity estimates for the southern Rocky Mountain region, including the Front Range and Rio Grande Rift. In addition, this work will provide a critical set of measurements to which to refer future project measurements. Thus, if a future moderate or large earthquake were to occur in Colorado, the earthquake size and faulting geometry might be accurately determined by a second set of measurements.

INTRODUCTION

The Colorado Front Range and the Rio Grande Rift have experienced moderate seismic activity in recent times. The largest known historical earthquake in Colorado was the November 8, 1882 earthquake, with an estimated Richter magnitude of $6.2 < M_L < 6.6$, and located in north-central Colorado [Spence et al., 1996]. Among the best documented earthquakes in Colorado is the 2001 Trinidad swarm [Meremonte et al, 2002], and other thoroughly studied events include those induced by the disposal of waste fluids at the Rocky Mountain Arsenal near Denver [Evans, 1966; Healy et al., 1968; Herrmann, 1981] and secondary oil recovery in western Colorado at the Rangely oil field [Gibbs et al., 1973]. The largest instrumentally recorded natural earthquake in Colorado was a magnitude 5.5 event in 1960 which occurred near Ridgeway in southwest Colorado [Talley and Cloud, 1962]. Fault-scarp data on Sangre de Cristo Fault Zone in south-central Colorado suggests past earthquakes of magnitude 7 to 7.3 [McCalpin, 1986]. The base map (Figure 1) of Quaternary faults in Colorado [after Widmann et al, 1998] documents 92 different faults with Quaternary offset, including 8 major faults with Holocene displacement. One of these Holocene faults, the Williams Fork Mountains Fault, is just 50 mi west of the Denver metropolitan area and is capable of producing a moment magnitude 6.75 earthquake [Unruh et al., 1993]. Indeed, a significant swarm of seismicity of magnitudes 2.8 - 4.6 took place during a 3 1/2 week period during our survey in September of 2001 southwest of Trinidad, CO [Meremonte et al, 2002], which has in part led to a major resurgence of interest in the seismic hazard of the region.

Despite this evidence for past and present earthquake activity, the nature and rates of the tectonic processes responsible for creating this seismic hazard are not well understood. The contemporary

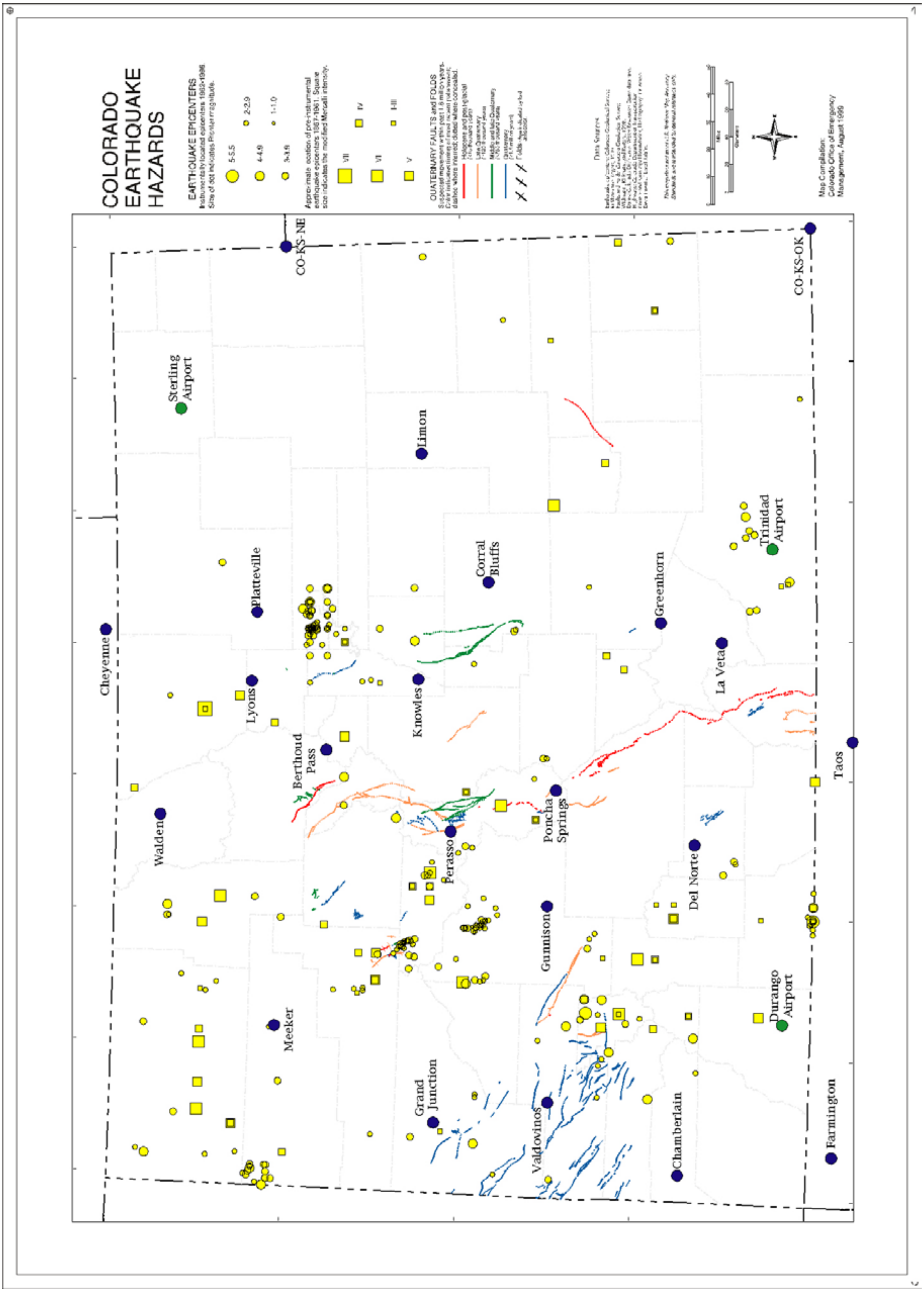

Figure 1. Location of 2001 GPS Measurements (Fault Data after Widmann et al, 1998).

crustal stress regime in Colorado includes extension along a roughly northeast-oriented axis [Bott and Wong, 1995]. A possible mechanism for seismogenesis in the area is reactivation of existing faults, which are oriented favorably to the contemporary stress field, i.e. oriented west-northwest to northwest. Focal mechanisms of two of the earthquakes related to the Rocky Mountain Arsenal fluid injection have indicated normal (or extensional) fault movement locally in the northern Front Range region [Wong, 1986]. In the Rio Grande Rift zone, recent VLBI results [Argus and Gordon, 1996] indicate larger rate than geologic and ground geodetic measurements. Their estimates are on the order of 4-5 mm/yr, whereas estimates from the geology of the Albuquerque basin [Woodward, 1977] are on the order of 0.3 mm/yr, extension estimates from Cordell [1982] are 0.5 to 1 mm/yr, and trilateration data yields 1 mm/yr (Savage et al., 1980. The large variations in estimated rate result in wide uncertainties in estimating recurrence rates for earthquakes in the region. No modern deformation rates have been measured for the Front Range area. Knowledge of the nature and orientation of tectonic stress regimes within the southern Rockies will provide a basis for assessing the capability of faults to slip seismically and produce earthquakes and is a useful tool to evaluate potential seismic hazard in the region.

We are seeking to answer fundamental questions about the mountains in our back yard: Are they a dead mountain range? Alternatively, are the Rockies currently undergoing tectonic uplift, or subsidence and extension? Are the numerous Quaternary faults in Colorado a significant source of seismic hazard? Seismic tomography work [Sheehan et. al, 1995] shows anomalously slow seismic velocities in the upper mantle beneath the Rockies, consistent with the northward propagation of the Rio Grande Rift. Our results will allow us to test this hypothesis and better determine any surface expression of these mantle anomalies.

BODY OF PAPER

Outline of the Method

One way of obtaining crustal deformation rates is by comparing two sets of high-precision GPS measurements taken at the same network of points at two different times. The total errors in the two sets of measurements must be small enough so that the accumulated movement within the network during the intervening time period can overcome the noise. Our aims were two-fold: to acquire a new set of measurements statewide in 2001 that are of the highest possible precision using state of the art acquisition and processing techniques using a previously established set of control points: and to reprocess data measured on these points 10 years earlier to high enough precision to obtain an estimate of crustal strain without requiring a future remeasurement.
The level of precision necessary to use GPS for tectonic studies can only be obtained by extensive post-processing of data that were obtained using proper field procedures, and if the GPS satellite orbits can be post-processed to a high level of precision using a global tracking network. This tracking network has been in place and orbits calculated by the International GPS Service since 1992, but as the original measurements were done in 1991, we are trying to devise our own strategy to improve the accuracy of the satellite orbits. Furthermore, the 1991 data were acquired by the National Geodetic Survey for geodetic data definition and not tectonic studies, so the potential accuracy of the reprocessed results is reduced even further, adding to the challenge

of obtaining an immediate result. Given the anticipated low rates of strain in the region, we must reduce the present errors in the reprocessed data by two orders of magnitude from its current levels if a meaningful result is to be calculated.

2001 Field Measurements and Processing

Twenty-six control points of the National Geodetic Survey's High Accuracy Reference Network, HARN, were occupied in August and September of 2001 using Trimble and Ashtech geodetic quality GPS receivers. These points were chosen on the basis of their geographic distribution, their physical stability and potential longevity, their security during extended multi-day and night recording sessions, and the availability of data recorded at these sites in 1991. The data were acquired by personnel from CIRES, the National Geodetic Survey, the Colorado Department of Transportation, and the Bureau of Land Management under our direction. Three stations (Sterling, Trinidad, and Durango Airports) were operated continuously, while the others were operated for between 36 and 72 hours. Specially designed low profile, low eccentricity antenna mounts were used wherever conditions permitted, and antenna positioning was carefully monitored to millimeter precision to assure stability during the sessions.

Processing was done using Bernese version 4.2 software [Rothatcher and Mervart, 2001], using high-precision satellite orbits recalculated by the International GPS Service to 20 centimeter precision and carrier phase double-differencing ambiguity resolution. Ionospheric delays are calculated using dual frequencies, earth tides and pole positions are precisely accounted for, and tropospheric effects are statistically modeled. Positions are calculated with respect to four permanent tracking stations (Colorado Springs, central New Mexico, Southern California, and western Canada) operated by the International GPS Service (IGS), in order to tie into the ITRF97 global reference frame.

The resulting coordinates for the network and their estimated errors are shown in Table 1, and the geographical locations are shown in Figure 1 along with the Colorado Earthquake Hazards Map [Colorado Office of Emergency Management, 1999]. The errors, currently on the order of 1 centimeter, will require at least 5 years before a remeasurement of the network can yield an accurate crustal strain rate with errors < 2mm/yr. Efforts are ongoing to reduce the errors by manual identification of outliers and cycle-slips, and we expect to reduce the overall uncertainty by a factor of 2 or more in the near future.

Use of 1991 GPS Measurements to Determine Regional Seismogenic Strain

HARN (High Accuracy Regional Network) was established in 1991 for the purpose of adjusting the geodetic mapping datum of North America to a space-based standard. There are 165 of these benchmarks installed in Colorado using various high-stability monumentation techniques depending on local geology. Per standard NGS practice, the 1991 measurements consist of relatively short 3-6 hour sessions, resulting in a roughly 15 cm precision level, more than adequate for their purpose of continental scale mapping datum adjustment. Unfortunately, the International GPS Service was not established until 1992, and therefore precise reprocessed satellite orbits are unavailable for reprocessing of the 1991 data. The use of broadcast orbits introduces an uncertainainty of 20 meters into the satellite positions which maps into uncertainties of tens of centimeters in carrier phase reprocessed positions. We are currently experimenting with techniques to improve the 1991 satellite orbits, which, when combined with state-of-the-art point positioning algorithms may yield results of sufficient precision to calculate strain rates given the 10 year period between the two measurements. However, it is likely that even if we are successful, the errors in the 1991 positions will be at least 5 cm., which will obscure any tectonic signal of less than 7 mm/year. Given the anticipated extension rates are less than 5 mm/year, it is unlikely that we will be able to definitively determine any local effects, although some regional results may be possible due to some extended measurements taken at a few stations in 1991.

SUMMARY AND CONCLUSION

Currently we only have obtained one set of GPS measurements of sufficient precision for use in calculating crustal strain. Reprocessing of both the 2001 and 1991 is ongoing in an attempt to reduce the position errors in order to resolve low levels of crustal strain postulated to exist in the southern Rockies without a future remeasurement of the 2001 network. Errors (Table 1) in the processed data are currently on the order of 1-2 cm., which given the smaller than 5 mm/yr deformation rates we must resolve, will require a 10 year remeasurement interval to obtain a meaningful strain rate if no improvement in the 2001 positions is obtained. In any case, repeating the 2001 measurements in five to ten years time will likely provide a precise calculation of the strain field in Colorado and quantify seismic hazard in the area.

The establishment of dense networks of new monumentation in areas of higher seismic hazard, such as the Rio Grande Rift or Trinidad would be of high value in addressing local hazard and tectonic concerns. These networks could be occupied periodically, in campaign style, or continuously if the means were provided

Further analysis of archived continuous IGS and other GPS data in the Rockies may be useful on a regional level. The IGS network was being continuously densified during the late 1990's and sufficient continuous data may exist to determine regional strain rates over the last 4 – 6 years. Other potential sources of continuous GPS data such as the NGS and UNAVCO (University NAVSTAR Consortium) have been identified and we are currently evaluating the suitability of the data for use.

A new project for seismic hazard evaluation using archived seismological data in the Rocky Mountains has been funded by the National Earthquake Hazard Reduction Program and work is in the initial stages. We hope to use these data, acquired during the Rocky Mountain Front Experiment [Lerner-Lam et al, 1998] in 1992 using broadband seismometers, will be reevaluated to determine magnitudes and locations for all seismic events of Mb>2.0 and source parameters for events of Mb>3.5 during the experiment. This will allow us to characterize existing and newly identified faults in proximity to the very densely urbanized Front Range corridor of Colorado (Denver/Colorado Springs/Ft. Collins). We will produce a database available for crustal attenuation studies to help assess wave amplitudes and seismic hazard, and input parameters for the National Seismic Hazard and Risk Maps including seismicity and source parameters (focal mechanisms, stress drop).

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Table 1. Processed 2001 GPS HARN Control Point Coordinates and Precisions

Site	Session Length	Latitude	Longitude	Height (m)	Nσ (cm)	Eσ (cm)	Uσ (cm)
STERLING AIRP.	Continuous	40.6216	-103.2666	1208.938	0.60	1.14	1.48
TRINIDAD AIRP.	Continuous	37.2609	-104.3377	1732.143	0.56	1.20	1.92
DURANGO AIRP.	Continuous	37.1588	-107.7528	2013.348	0.44	0.60	0.98
DEL NORTE	36 Hours	37.6828	-106.4988	2439.448	1.28	3.14	6.74
LAVETA AIRP.	36 Hours	37.5248	-105.0016	2138.670	1.00	2.36	4.78
KNOWLES	36 Hours	39.2522	-105.2392	1990.856	1.54	3.92	7.54
PERASSO	36 Hours	39.0838	-106.3060	2797.794	2.38	6.44	12.64
TAOS AIRPORT	72 Hours	36.4603	-105.6690	2137.716	1.16	2.90	6.18
GREENHORN	36 Hours	37.8889	-104.8560	1864.335	1.66	4.02	8.26
PONCHA SPRINGS	36 Hours	38.5170	-106.0712	2261.234	2.78	6.86	14.22
LIMON AIRPORT	36 Hours	39.2684	-103.6641	1609.166	1.22	2.94	5.94
CORRAL BLUFFS	72 Hours	38.8698	-104.5934	2051.130	1.00	2.42	4.86
CLAYTON AIRP	24 Hours	36.4451	-103.1560	1491.375	2.46	6.86	14.16
BERTHOUD PASS	36 Hours	39.7989	-105.7778	3436.839	1.62	3.80	8.00
GUNNISON	36 Hours	38.5388	-106.9267	2325.805	1.98	5.14	10.64
CO-KS-NE Boundary	36 Hours	40.0032	-102.0518	1026.168	2.30	5.32	12.18
FARMINGTON AIRP	24 Hours	36.7400	-108.2195	1654.654	2.08	5.46	10.84
GRAND JCT AIRP	48 Hours	39.1063	-108.5337	1435.100	1.02	2.50	4.10
MEEKER AIRPORT	72 Hours	40.0420	-107.8929	1920.077	0.78	1.12	2.14
CHEYENNE GPSBASE	72 Hours	41.1341	-104.8672	1867.411	0.74	0.60	1.46
CHAMBERLAIN	36 Hours	37.7183	-108.8531	2004.180	1.24	3.38	6.94
VALDOVINOS	36 Hours	38.4959	-108.3515	2496.140	1.28	3.36	6.48
LYONS	36 Hours	40.2283	-105.2689	1655.603	1.16	2.52	4.96
PLATTEVILLE	72 Hours	40.1828	-104.7263	1501.276	0.92	1.96	3.58
WALDEN AIRP.	36 Hours	40.7440	-106.2805	2471.924	1.36	2.52	5.52
CO-OK-KS Boundary	36 Hours	36.9931	-102.0421	1098.001	4.02	9.06	18.90
PIETOWN IGS	Continuous	34.3015	-108.1189	2347.778	0.44	0.44	0.62
PINYON #1 IGS	Continuous	33.6122	-116.4582	1256.163	0.44	0.48	0.26
SCHREIVER IGS	Continuous	38.8031	-104.5246	1911.303	0.46	0.98	0.56
YELLOWKNIFE IGS	Continuous	62.4809	-114.4807	1208.938	0.22	0.80	0.22

APPENDIX I: SUPPLEMENTAL READING LIST

Listed below are additional references that were not cited in the text:

Colman, S. M., Morphology and age of fault scarps in the Rio Grande Rift, South-Central Colorado, in *Contributions to Colorado Seismicity and Tectonics - A 1986 Update*, Colorado Geological Survey Special Publication 28, p. 43-58, 1986.

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CONTEMPORARY SEISMICITY, 1983 - 1993, AND ITS IMPLICATIONS TO SEISMIC HAZARD IN THE CENTRAL FRONT RANGE, COLORADO

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Key Terms: seismicity, focal mechanisms, potentially active faults, tectonic stress, seismic hazard

ABSTRACT

The occurrence of the 1882 **M** 6.6 earthquake suggests that there may be a significant, albeit poorly quantified, seismic hazard in the Front Range of Colorado. In an effort to better characterize this hazard in the central Front Range, we have used a joint hypocenter-velocity inversion to improve hypocentral locations and derive a velocity model from a well-recorded set of microearthquakes (Richter magnitude [M_L] ≤ 3.3). The dataset consists of more than 1,100 events recorded by Microgeophysics Corporation (MGC) from 1983 to 1993 for the Denver Water Department. About 50 stations from the MGC network were used in this study. A 1D joint hypocenter-velocity inversion was first used to find the best estimate 1D velocity model for the region and the dataset. These results were then used as a starting model for a 3D inversion.

The majority of the more than 800 relocated earthquakes show no definitive association with the surface traces of mapped "potentially active" faults (Miocene and younger), although some events cluster at the ends of a few of these faults and sometimes along them. However, seismicity occurring in the vicinity of, and not apparently along, late Quaternary faults is the typical pattern observed throughout much of the interior western U.S. A significant aspect of the central Front Range seismicity is its persistent, moderate level of activity relative to many other portions of the Rocky Mountain region. Similar seismically active areas in the western U.S. are generally confined to areas of known late Quaternary faulting. In general, focal mechanisms are consistent with normal and strike-slip faulting in the western portion of the study area and reverse and strike-slip faulting in the eastern portion along the N- to NW-striking faults.

Although paleoseismic investigations in the past 20 years indicate that displacement along many of the faults (e.g., Floyd Hill, Oil Creek faults) has not occurred in the past 100,000 years or so, we believe their earthquake potential still remains unresolved given our observations of the contemporary seismicity and also the paucity of late Quaternary sediments in the region. Three possible explanations that would be consistent with the paleoseismic evidence are (1) deep-seated (> 15-18 km) rupture resulting in little to no surface displacement along the faults, (2)

long recurrence intervals of several tens of thousands to more than 100,000 years, or (3) the Miocene and younger faults are not seismogenic and that earthquakes like 1882 occur on buried or yet undiscovered faults.

INTRODUCTION

Historically, central Colorado has experienced only low levels of naturally-occurring seismicity, with the 1882 earthquake of moment magnitude (**M**) 6.6 ± 0.6 being the most significant event to shake the Denver metropolitan area (Kirkham and Rogers, 1985, 1986; Spence *et al.*, 1996) (Figure 1). This observation excludes the Rocky Mountain Arsenal (RMA) induced earthquakes of the 1960's which began shortly after the injection of waste fluid into the crust (Healy *et al.*, 1968; Major and Simon, 1968; Herrmann *et al.*, 1981; Hsieh and Bredehoeft, 1981). Twelve events in the sequence of body-wave magnitude (m_b) 4 to 5 generated significant ground shaking in Denver (Modified Mercalli [MM] intensity ≥ VI) (Kirkham and Rogers, 1985). With the exception of the 1882 event, the largest known earthquake to have occurred within the Front Range was a m_b 4.0 earthquake that occurred on 25 December 1994 near Castle Rock at a depth of 23.5 km (Kirkham and Rogers, 2000). This earthquake shook an area of 1,700 km^2 along the boundary between the Front Range and the Great Plains (see further discussion).

In contrast to the sparse historical seismicity, microearthquake monitoring by Microgeophysics Corporation (MGC) in the Front Range, west and southwest of Denver from 1983 to 1993 for the Denver Water Department (DWD), has revealed a surprising moderate level of seismicity (Richter magnitude [M_L] ≤ 3.3). About 2,300 local events (defined as S-P time ≤ 3 sec) were detected during the 10-year period from which about 1,000 events were located by MGC (1994). In some instances, planar steeply-dipping clusters of seismicity appear to be associated with the surface traces and subsurface projections of Miocene and younger faults identified by Kirkham and Rogers (1981) as "potentially active" faults (Unruh *et al.*, 1996). In the study area, the most prominent of these faults include the Golden, Floyd Hill, Kennedy Gulch, Ute Pass, Oil Creek and Rampart Range faults. These faults appear to be characterized by very low activity rates with long recurrence intervals of more than tens of thousands of years (Widmann *et al.*, 1998).

Unruh *et al.* (1998) suggest that the slow extensional deformation in the southern Rocky Mountains is driven by lithospheric gravitational potential energy and is accommodated locally by reactivation of pre-existing crustal faults. Although the southern Rocky Mountains are characterized by high potential energy, Jones *et al.* (1996) suggested that the region is deforming relatively slowly because of the strong underlying lithosphere.

The study of intraplate microseismicity and its relation to the reactivation of older faults is important for the Front Range because of the implications to seismic hazard. The 1882 earthquake is now thought to have occurred somewhere along the northern Front Range (Kirkham and Rogers, 1985, 1986; Spence *et al.*, 1996), and thus the potential for similar-sized events may exist elsewhere along the Front Range. If we can identify which faults might be reactivated, even at the microearthquake level, our understanding of any potential earthquake threat to the rapidly growing central Front Range region can be improved.

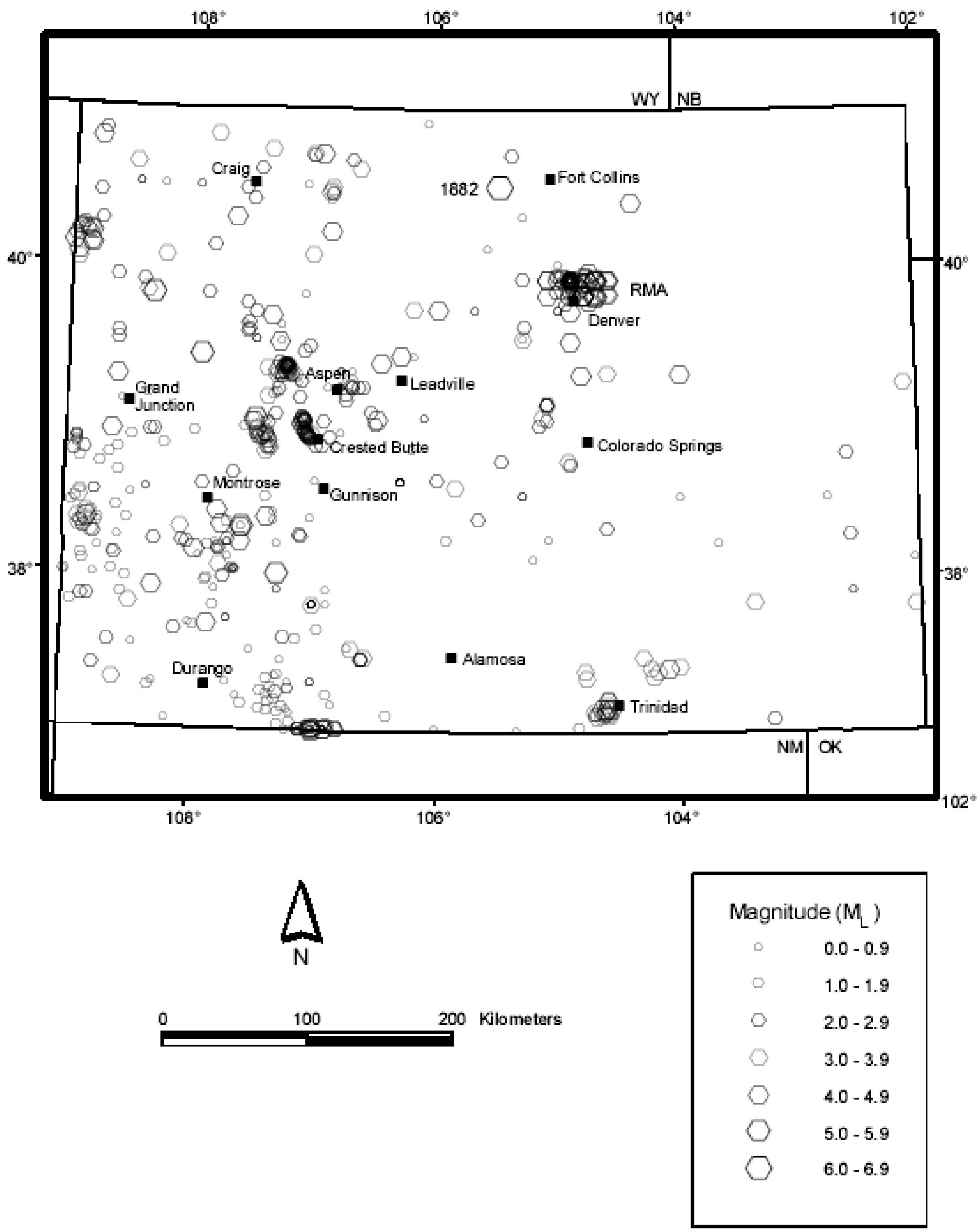

Figure 1. Historical seismicity (1870-2003) in Colorado. Location accuracy varies greatly over the state with large uncertainties (> 50 km) possible for pre-instrumental (pre-1960's) earthquakes.

Thus, the objectives of this study were to (1) improve the locations of earthquakes recorded by MGC using a joint-hypocenter velocity inversion, (2) better characterize the local crustal velocity structure, and (3) compute reliable focal mechanisms. Based on the improved hypocentral locations, possible associations between the earthquakes and the mapped Miocene and younger faults will be evaluated. The style of faulting determined from focal mechanisms can help characterize the regional state of stress in this transition zone and thus help in identifying which faults might be preferentially reactivated. Based on these results, the seismotectonic setting of the central Front Range and hopefully the seismic hazard for the region can be better characterized. The following paper summarizes the approach, the results of the analyses, and our interpretations of those results. A more complete description of this study, which describes in better detail the inversion for crustal velocity structure, is contained in Bott *et al.* (2003).

APPROACH

Although the number of MGC stations and the network configuration varied throughout the 1983 to 1993 monitoring period (MGC, 1994), in general, the network covered the portion of the Front Range between Boulder in the north and Colorado Springs to the south and from Lakewood westward for a distance of about 60 km (Figure 2). More than 50 station locations were occupied over the 10-year period, but the station configuration changed with only about 20 to 30 stations operational at any one time.

Only limited information gathered by MGC from the Front Range network operations has been published (Butler and Nicholl, 1986). However, the whole Front Range data set has been made available recently by DWD for use in seismic hazard studies of other facilities within central Colorado (e.g., Unruh *et al.*, 1996; Bott *et al.*, 1996) and this has renewed interest in this important data set.

Joint Hypocenter-Velocity Inversion

In the routine processing of earthquake locations, MGC used the program HYPO71 (Lee and Lahr, 1972), and a velocity model consisting of a 1.95 km-thick layer with a P-wave velocity (V_P) of 5.1 km/sec over a half-space velocity of 5.95 km/sec. The velocity model was originally derived from blast data. Arrival-time data from a different set of blasts were used to verify and constrain the velocities in the uppermost layers of the crust. In this study, a 1D joint hypocenter-velocity inversion was first performed to determine the minimum V_P model (FR1D), using the program VELEST (Kissling, 1988) and the 190 best-located events. Velocity layers were fixed in the near surface down to a depth of 4 km based on the blast results. Layer thickness varied from 1 to 2 km thick in the upper crust increasing to 4 to 6 km thick in the mid-crust. The best estimate 1D minimum model obtained was similar to the models of MGC and Prodehl and Lipman (1989) for the upper 10 km of the crust. However, slightly higher V_P values (6.15 to 6.25 km/sec) were found between the depths of 10 and 26 km. No rays traversed layers below a depth of 26 km and so the velocities at these depths were not constrained. Based on the arrival time data, the V_P/V_S ratio is 1.71 (shear-wave velocity V_S), lower than the 1.87 originally used by MGC. The V_S was calculated assuming this ratio for the upper crust.

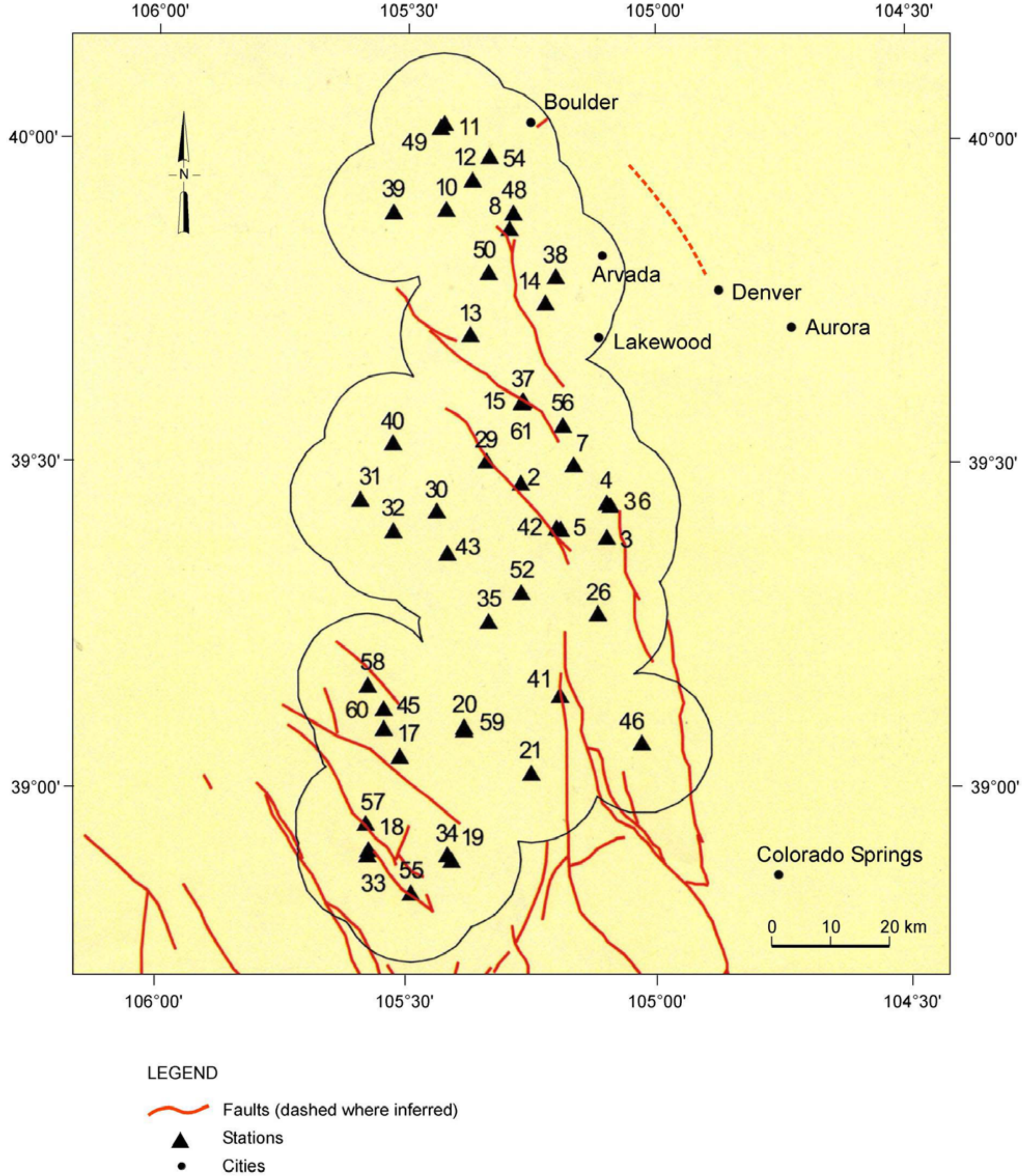

Figure 2. Front Range stations operating for more than 1 year and 12-km arc around each station within which all M_L 1.0 events are thought to have been detected. Miocene and younger faults are from Kirkham and Rogers (1981).

The FR1D model incorporates station corrections, which were compared with the surface geology. Most stations located on the 1.08 billion-year-old Pikes Peak batholith (Smith *et al.*, 1999) have the highest positive station corrections, implying slower velocities than the FR1D model predicts. In contrast, those stations located on older Precambrian metamorphic rocks (1.4 to 1.8 billion-years-old) have negative station corrections, implying faster velocities than FR1D predicts. MGC also used station corrections, which generally reflect the difference in station elevation and the elevation datum (2.5 km above mean sea level) since topography is not accounted for in HYPO71. Station elevations are accounted for in FR1D, and so the calculated station corrections should only account for the difference in the underlying rock velocities. Earthquake relocations of the best-located 190 events used in the 1D inversion moved on the order of 2 to 3 km from their original locations and a tightening of clusters of events was observed. The velocity model was fixed and the inversion repeated with all station elevations set to zero, to produce a set of station delays that could be used with program HYPO71 (velocity model referred to as FR1D-H71). Relocations of 11 known blasts using FR1D-H71 showed improvement on average of 0.9 km over locations using the MGC velocity model when compared to their known locations.

All events thought to be earthquakes were relocated with the new velocity model (FR1D-H71). An improvement was observed in the locations based on the average rms and location quality when compared to those computed using the MGC model. Earthquakes in the northern part of the network generally occur at shallow depths (< 15 km) and are diffusely distributed, whereas earthquakes in the south mostly occurred at depths greater than 8 km and sometimes in clusters or as swarms. In the north, the geology is comprised of Precambrian metamorphic rocks, whereas to the south, the network is underlain by the younger anorogenic granite of the Pikes Peak batholith. Structurally, the metamorphic rocks under the northern part of the network are cross-cut by numerous NW-SE-striking faults, whereas the batholith is cut by fewer faults and these generally strike N-S or are ring fractures associated with the emplacement of plutons within the batholith.

The 190 relocated earthquakes and the FR1D velocity model were used as initial input for a 3D joint hypocenter-velocity inversion using the program FDTOMO (Benz *et al.*, 1996). Crustal velocities were determined for the middle and upper crust, down to about 26 km in some areas. Moho refractions are not found at the epicentral distances (less than 100 km) used in this data set. The relocated hypocenters were not too different from the locations using FR1D but are generally deeper, with average shifts of 1.0 km horizontally and 1.4 km vertically. Relocations of the same set of known blasts showed significant improvement with the average location being within 0.5 km of the actual site. Final relocations of more than 800 earthquakes using FR3D are shown in Figure 3.

The tomographic velocity model provides an interesting image beneath the Front Range, with a region of slightly lower velocity directly under the outcrop of the Pikes Peak batholith. This lower velocity area may define the shape of the batholith at depth. This region also correlates well with the area for which positive station corrections were determined using FR1D. It is observed that the microseismicity in the vicinity of the Pikes Peak batholith is restricted to clustering of events around and below the low-velocity region. Significant deformation may not be occurring internally within the batholith but may be localized along distinct fault zones around its edges and below it.

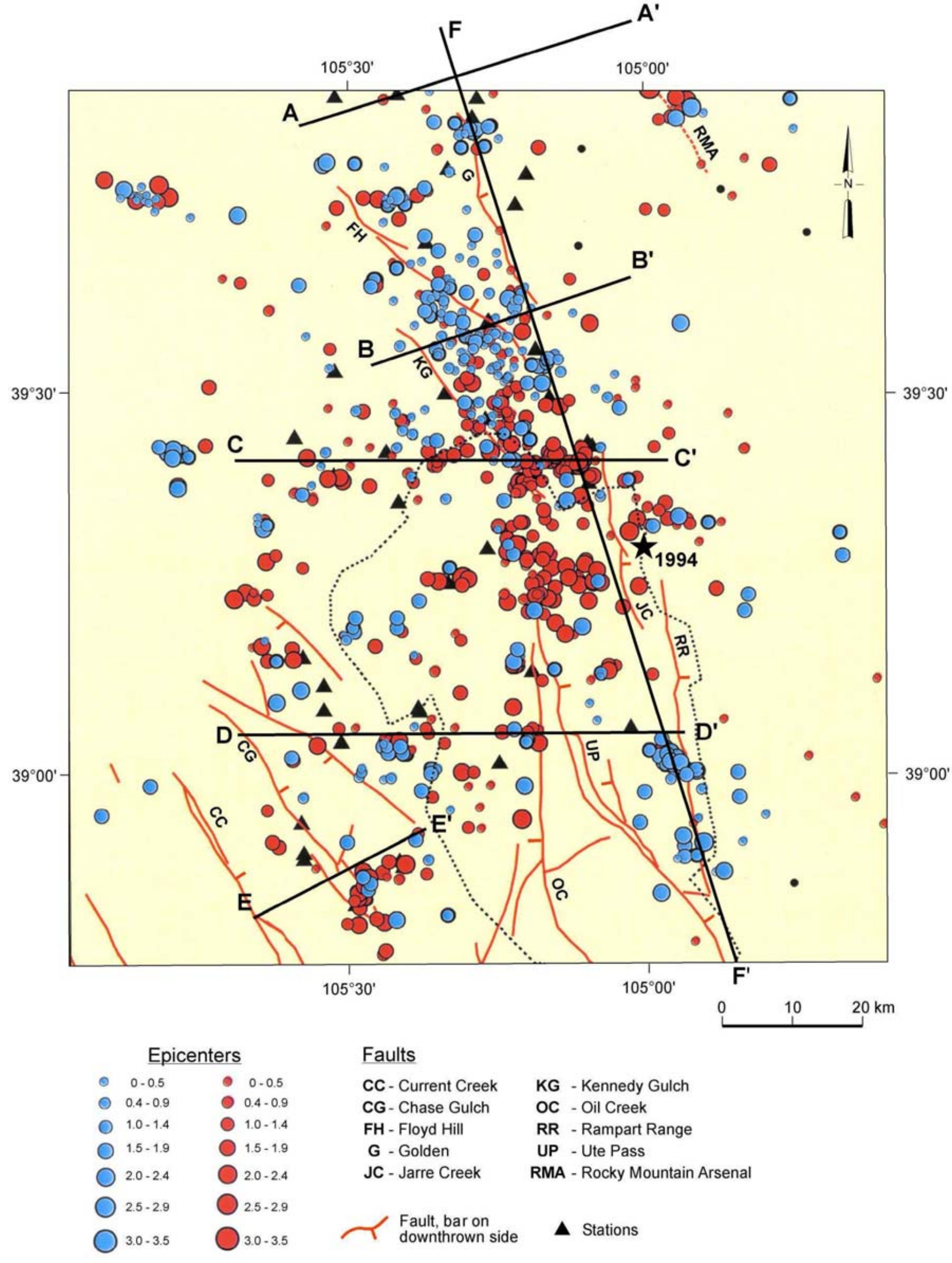

Figure 3. Relocations of all tectonic earthquakes using FR3D showing shallow (≤ 8 km) events in blue and deep events (> 8 km) in red. Dotted line is the boundary of the Pikes Peak batholith. Unnamed faults not labeled. Cross-sections are shown in Figure 5. The location of the 25 December 1994 m_b 4.0 Castle Rock earthquake is also shown.

In the northern part of the network, the microseismicity is diffusely distributed both in space (Figure 3) and time. The earthquakes could be occurring on the many small faults that crosscut the metamorphic rocks in this region. Thus, the northern and southern regions appear to be deforming differently possibly due to their differing geology, and the location and orientation of pre-existing faults, despite the fact that both regions probably are being subjected to the same extensional tectonic stress field.

An area of slightly higher velocity lies directly below the Pikes Peak batholith (> 6 km), which we believe is defined by the slightly lower velocities as described earlier. We speculate that this pattern could be revealing the existence of deep thrust structures under the Front Range, or a mafic igneous body. The latter is consistent with the model for the genesis of the A-type granites suggested for the Pikes Peak batholith (Smith *et al.*, 1999) and is our preferred model. This hypothesis is also supported by the gravity data as other authors have suggested that the slight gravity high above the batholith indicates that the normally low-density granite (and thus an expected negative gravity anomaly) is somehow compensated for at depth by a denser rock mass.

Focal Mechanisms

An objective in this study was to use the improved locations to better determine single-event focal mechanisms. Focal mechanisms were computed for the best-located events (A and B quality locations) using the 3D velocity model and the program FPFIT (Reasenberg and Oppenheimer, 1985). Only mechanisms with at least 10 first motions were calculated.

A known problem with the data set was that several stations were believed to have polarities that were reversed from the standard assumption that an “up” motion was compressional and a “down” motion was dilatational. Unfortunately, the dates of station reversals were not completely documented and so we attempted to reconstruct the network station polarity history using the available data. To further interpret this data set, it was necessary to select a subset of the better-constrained events and stable solutions. To accomplish this, events with focal depths less than 5 km were removed as these depths are not as well constrained and thus probably not as accurate as those of the deeper events, based on station spacing.

Due to the difficulty of assessing the station polarity history by sifting through the thousands of original smoked paper records to identify known blasts, another approach was employed to utilize the available data. Stations recording the known and suspected blast events that did not have compressional first motions were noted. A list of suspect stations and their estimated time periods of reversal was compiled and used as input to the FPFIT program for a second focal mechanism determination. The original smoked paper records were checked for a few selected months to verify some of the station reversals.

The FPFIT program was rerun using the 3D-velocity model incorporating the proposed station reversal history and also using the 1D and the original MGC models. Focal mechanisms for events which appeared relatively stable despite a change in velocity model, and the incorporation of the station reversals, were selected to represent the final subset of data.

Focal mechanisms were also calculated for several events for a range of fixed focal depths to evaluate their sensitivity to depth. Several events, chosen to represent a range of faulting style and focal depth, were relocated with the focal depth fixed in 1-km increments from 5 to 25 km. The 1D velocity model was used for these analyses due to speed of running HYPO71. It was noted that for each of these events, their rms error occurred in a pronounced minimum in the rms error versus depth curve indicating that the actual focal depth was stable. The focal mechanisms were also found to be relatively stable for most of the events over the entire depth range, except in a few cases where the mechanism appeared to be affected for depths close to a velocity interface. The final filtered data set consists of 25 focal mechanisms: 12 normal, 7 reverse, 4 strike-slip, and 2 oblique-slip solutions (Figure 4).

RESULTS AND DISCUSSION

The earthquake relocations using the 3D model differ from the starting MGC locations, but mostly in their depth. Based on the relocation of the 11 quarry blasts, we believe the epicentral locations are accurate to within about 0.5 km, at least in the central portion of the network. The location accuracy probably deteriorates toward the edge and outside of the region covered by the network. However, it is noted that the locations of earthquakes occurring in the vicinity of the RMA, which are outside the network coverage, appear to be much closer to the fault than the original MGC locations (Figure 3), which indicates an improvement using the 3D velocity model. The depths of the RMA events are probably not well resolved and a change in the velocity with depth could have a large effect. In our model, the velocity increases with depth, whereas the MGC model has a constant half-space velocity below the top 2 km of the crust. It is therefore not surprising that the relocated earthquakes are much deeper than the original MGC locations. We believe the relocated events within the bounds of the network have a focal depth accuracy of ± 2 km.

Distribution of Seismicity

Based on the final relocations, microearthquakes appear to be distributed across the region covered by the network (Figure 3). A large concentration of events occurs within a 60-km-long NNW-SSW-trending rectangular area bounded partially on the west by the Floyd Hill and Kennedy Gulch faults, and on the east by the Golden and Jarre Creek faults (Figure 3). The 1994 Castle Rock earthquake, which was located by MGC (Kirkham and Rogers, 2000), appears to have occurred along the eastern margin of the Pikes Peak batholith (Figure 3). On the eastern edge of the batholith, along the Rampart Range fault, a small cluster of shallow events is observed orientated NW-SE rather than N-S parallel to the fault (Figure 3). This lineation could be an artifact of the station locations since this cluster is located towards the edge of the network region or it could be in fact a NW-SE-striking unmapped fault or fault zone.

The depth distributions of the relocated earthquakes are shown in Figure 5. Events are shown within 5 km of the cross-section. The majority of earthquakes occur at depths of less than 15 km. Cross-section C-C' shows an interesting and significant increase in the maximum depth of events from west to east across the network. The deepest earthquakes occur at depths down to 27 km beneath the northern edge of the Pikes Peak batholith (Figures 3 and 5, Sections C-C' and F-F'). In the area underlain by the batholith, shallow earthquakes are fewer in number (Figure 5, Section D-D').

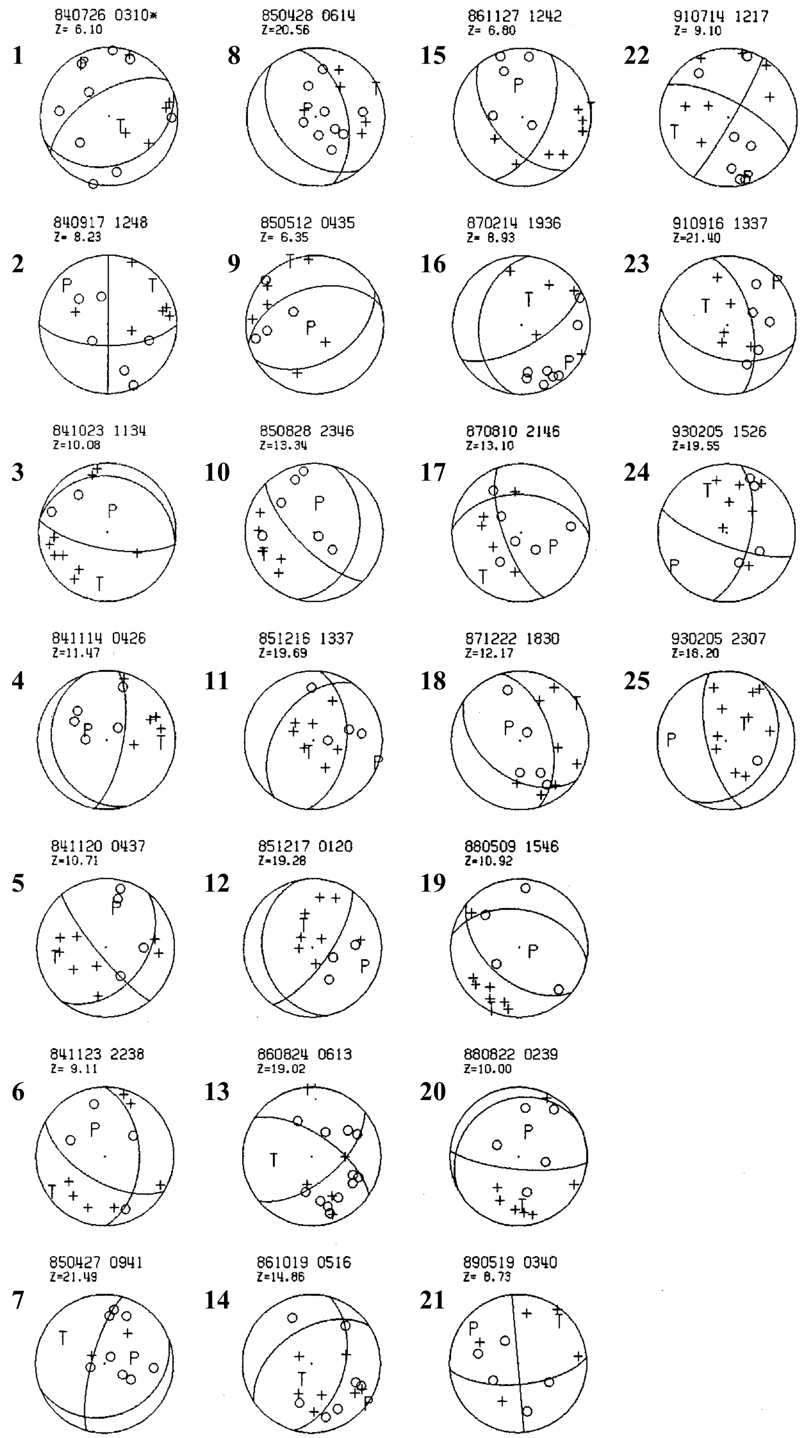

Figure 4. Focal mechanisms calculated in this study. Compressional and dilatational first motions indicated by plus symbols and circles, respectively. Orientations of pressure (P) and tension (T) axes are also shown.

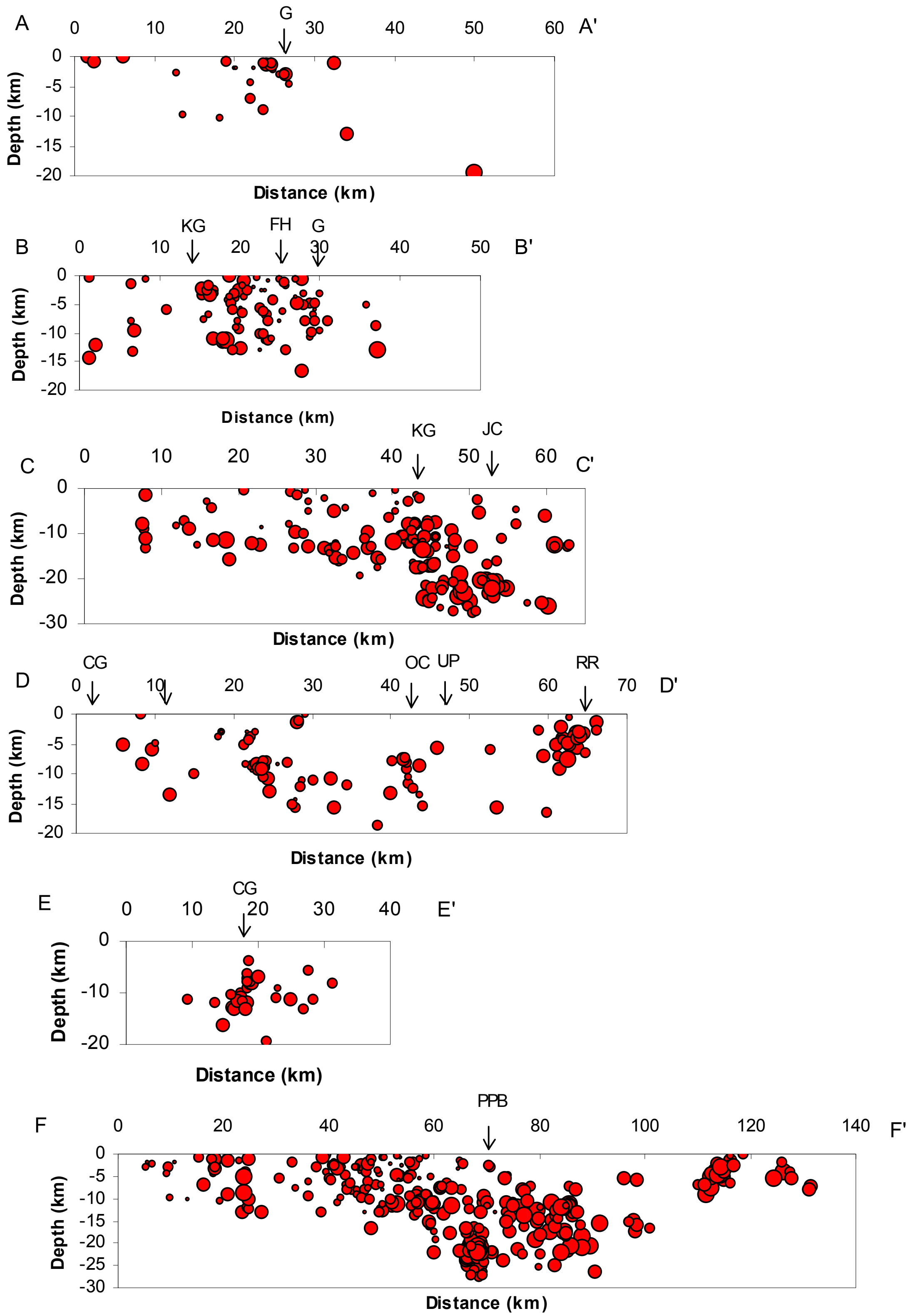

Figure 5. Cross-sections showing earthquake locations from the 3D velocity inversion. Locations of cross-sections and fault abbreviations shown on Figure 3. PPB in Section F-F' indicates northern edge of Pikes Peak batholith.

A NNW-SSE-trending cross-section parallel to the strike of the Front Range shows the same interesting distribution of earthquakes with depth (Figure 5, Section F-F'). In the north from a distance of about 0 to 70 km, the earthquakes are distributed somewhat uniformly throughout the top 15 km of the crust. Near the northern edge of the Pikes Peak batholith, a sudden increase in the depth of events is observed with most earthquakes occurring at depths greater than about 8 km to the south. The earthquakes then shallow towards the southern end of the network. Two distinct clusters are observed in Section F-F' (Figure 5) in the southern half of the network: one deep concentration just north of the batholith and one very shallow cluster at an along-strike distance of 100 to 120 km.

Earthquakes in the interior intraplate portions of the western U.S. are generally confined to the top 15 to 20 km of the brittle crust (Smith and Bruhn, 1984; Wong and Chapman, 1990). The observed unusually deep events (> 20 km) indicate that a relatively cold thick seismogenic crust may exist beneath portions of the central Front Range.

Relationship to Faulting and Focal Mechanisms

In general, there is no clear causal association between seismicity and known Miocene and younger faulting, although there may be a spatial coincidence in a few possible areas (Figure 3). Some relocated earthquakes appear to cluster at the northern end of the Golden fault, in the center of and at the southern end of the Floyd Hill fault, possibly along the southern portion of the Kennedy Gulch fault, at the northern end of the Ute Pass fault, just west of the center of the west-dipping Rampart Range, and at the southern end of the northeast-dipping Chase Gulch fault (Figure 3). This tendency for earthquakes to cluster at the ends of generally seismically quiescent faults has been observed in western Nevada (VanWormer and Ryall, 1980) and is consistent with the suggestion that stress accumulates at fault tips.

The apparent spatial coincidence of the epicenters with the ends of faults noted above, however, is not readily visible in cross-section, although the dips of these faults are poorly known. Possible exceptions are the northern end of the west-dipping Golden faults, the east-dipping Floyd Hill fault, the west-dipping Rampart Range fault, and possibly the east-dipping Chase Gulch fault (Figure 5).

Due to the distribution of stations, the focal mechanisms calculated in this study were confined to earthquakes located in the central portion of the network SSW of Lakewood (Figure 6). They exhibit a range of faulting styles including normal, reverse, strike-slip, and oblique slip (both normal and reverse) (Figure 4). Despite the variety of mechanisms, a pattern emerges that is not unexpected. West of the dashed line on Figure 6, the majority of focal mechanisms display normal faulting in response to NE-SW to E-W extension (minimum principal stress).

This tectonic stress field is consistent with the stress field inferred from other focal mechanisms in central Colorado (e.g., Wong, 1986; Bott and Wong, 1995). East of the dashed line, most of the focal mechanisms display predominantly reverse faulting in response to NW-SE to E-W-directed compression (maximum principal stress) as observed elsewhere in the Great Plains and Midcontinent (Zoback and Zoback, 1989). The principal stress directions for the four strike-slip mechanisms are consistent with a transition between the extensional stress field to the west and the compressional field to the east whereby the maximum principal stress rotates from a vertical

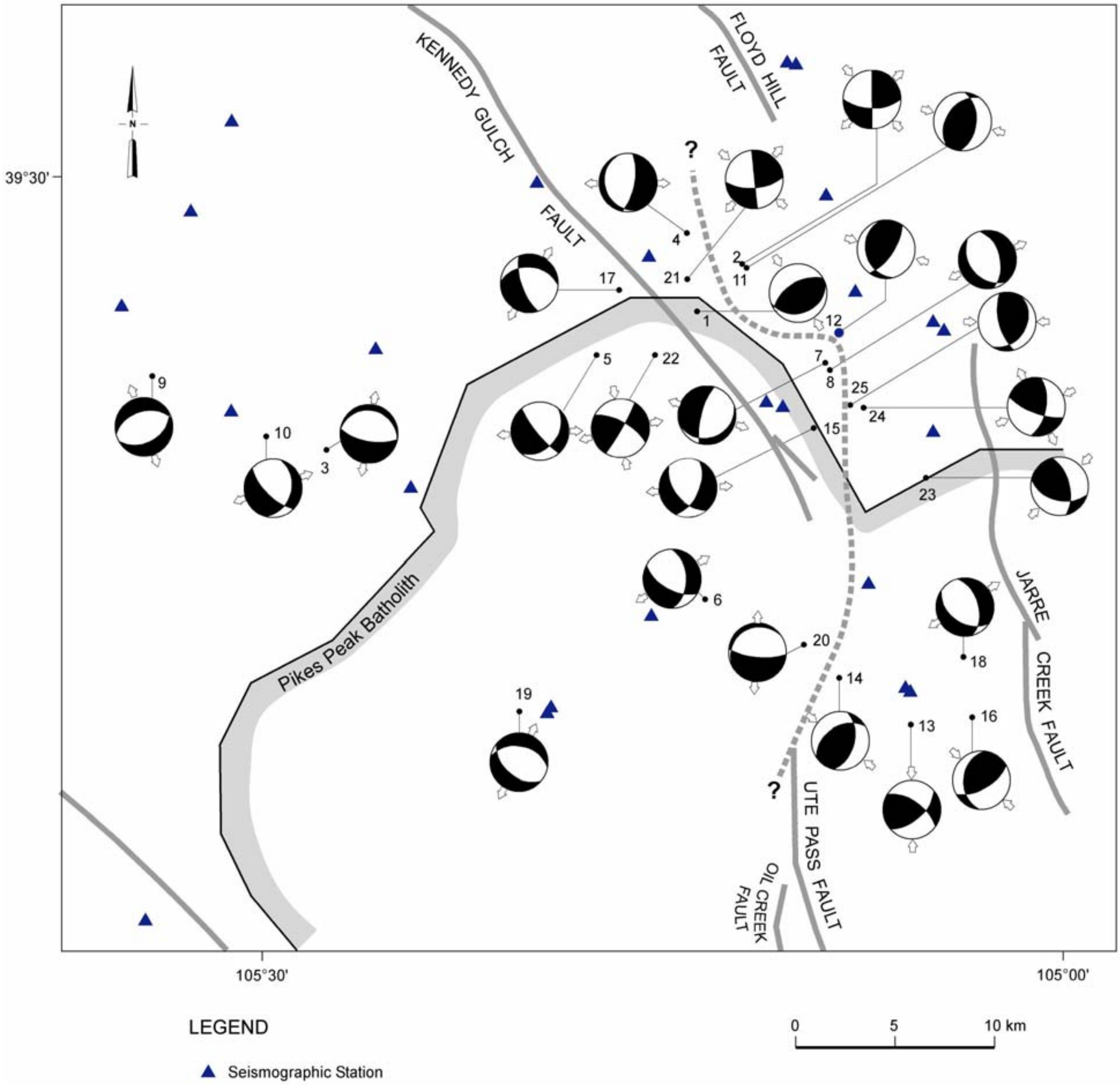

Figure 6. Map of schematic focal mechanisms calculated in this study. Dashed line represents the possible boundary between the extensional Rocky Mountains and the compressional Midcontinent tectonic stress fields. Inward and outward arrows indicate maximum and minimum principal stress directions, respectively. Shaded areas in focal mechanisms are compressional quadrants.

orientation to a horizontal position (Figure 6). The focal mechanism for the 1981 Conifer earthquake (Butler and Nicholl, 1986), which exhibits strike-slip faulting in response to an E-W maximum principal stress, is consistent with this transition. Normal faulting focal mechanisms along the RMA fault (Herrmann *et al.*, 1981) are east of the dashed line shown in Figure 6 suggesting that the transition from extension to compression may not run north-south consistently through the Front Range. The overall tectonic stress pattern is consistent with the large-scale pattern suggested by Zoback and Zoback (1989). An interesting but unresolved aspect of the reverse focal mechanisms is they are for events that appear to be occurring deeper (> 15 km) than the normal/strike-slip faulting earthquakes (< 13 km) to the west.

Implications to Seismic Hazards

A desire of this study was to better define the potential seismic hazards in the central Front Range by evaluating whether the relocated contemporary seismicity could be correlated with any normal/strike-slip faulting of the mapped potentially active faults identified by Kirkham and Rogers (1985), and whether focal mechanisms were consistent with these structures being seismogenic. In this regard, the results of these analyses are inconclusive. The majority of relocated seismicity does not occur in the vicinity of mapped faults. However, some earthquakes are observed near or along several faults particularly at their mapped terminations.

In an apparent contrast, paleoseismic studies suggest that the potentially active faults in the central Front Range are either inactive or very low activity faults. Investigations conducted by and on the behalf of DWD of the Floyd Hill (Dickson *et al.*, 1986), Kennedy Gulch (Yadon, 1986), Ken Caryl (Dickson and Paige, 1986), and Oil Creek faults (Friedman, 1986) have revealed no evidence for displacement along any of these structures in the past 80,000 to 100,000 years and in some cases, significantly longer. Circumstantial evidence for Quaternary slip is confined to the southern end of the Ute Pass fault (Dickson *et al.*, 1986).

Trenching studies by Dickson (1986) on the Rampart Range fault suggest that the most recent displacement took place between about 30,000 to 50,000 to as much as 600,000 years ago. No Quaternary displacement has been observed along the Jarre Canyon fault (Dickson *et al.*, 1986). The Golden fault has been the target of numerous paleoseismic investigations and the results have been ambiguous (Widmann *et al.*, 1998). Widmann *et al.* (1998) have classified the Ute Pass and Rampart Range faults (last displacement prior to 30,000 to 50,000 years BP) as Quaternary in age. Unruh *et al.* (1998) estimate vertical separation rates on faults in the Front Range from displaced Quaternary strata ranging from 0.001 to 0.1 mm/yr.

It is important to note that the near-absence of small magnitude seismicity along many of the faults in the central Front Range is in itself, not sufficient to rule out that the faults are seismogenic since fault-related microseismicity is the exception rather than the norm in the interior of the western U.S. (e.g., Smith and Arabasz, 1991; Wong and Olig, 1998). The observation that some seismicity is occurring at the ends of some faults, as observed in the Basin and Range Province, remains intriguing. Although partially a result of the detailed microearthquake monitoring, another more general aspect of the central Front Range is the moderate level of seismicity. In the authors' experience, such a level is observed only in regions of late-Quaternary faulting. Focal mechanisms are also consistent with the NW- to N-striking Miocene and younger faults observed in the Front Range being reactivated in normal, oblique-normal, and/or strike-slip displacement, in the extensional western portion of the region.

In summary, we believe, despite the sparse paleoseismic evidence, the Miocene and younger faults in the region may still be seismogenic. The faults could be very low activity faults with recurrence intervals on the order of several tens of thousands to greater than 100,000 years. This could explain the lack of evidence for surface displacement along most of the mapped faults in at least the past 100,000 years. Alternatively, as suggested by Spence *et al.* (1996), deep-seated rupture similar to the 24-km-deep 1985 **M** 5.5 Laramie Mountains earthquake may be a common mode of deformation in the Front Range. This is consistent with the presence of an approximately 25 km-thick seismogenic crust in portions of the central Front Range. An event like 1882 initiating rupture at a depth significantly greater than 15 km would likely result in no observable surface displacement accounting for the very sparse evidence for late-Quaternary activity on the Miocene and younger faults. A third possibility is that large earthquakes in the Front Range are occurring on buried or as yet undiscovered faults.

In light of both the microseismicity data and past paleoseismic studies, key questions with regards to seismic hazards in the central Front Range still remain unanswered: (1) what is the maximum sized earthquake that can occur; (2) what are the sources of such events; and (3) what are their recurrence intervals. Of primary relevance to addressing these questions is the 1882 **M** 6.6 earthquake. If the 1882 earthquake occurred in the northern Front Range as previously suggested, we know earthquakes of this magnitude, at a minimum, will likely occur in the future. Characterization of seismogenic structures whether they be any of the currently mapped Miocene and younger faults, unmapped or poorly studied faults, or buried faults remains our best hope for answering these questions.

ACKNOWLEDGEMENTS

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EXPANSIVE SOIL AND BEDROCK IN COLORADO

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Key Terms: expansive soil and bedrock, swelling soil and bedrock, bentonite, heaving bedrock, geologic hazards, damage, site exploration, mitigative designs

ABSTRACT

Expansive soil and bedrock constitute Colorado’s most costly geologic hazard in terms of damage to private and public facilities. These clay-bearing materials are widespread across Colorado, and are particularly common in the Jurassic, Cretaceous, and Tertiary formations and their derived, Quaternary soil deposits. Expansive soil has caused slight to significant damage to pavements, driveways and sidewalks; building walls, floors, and foundations; and water and sewer lines in many parts of the state. The extent and the severity of the hazard depends upon a number of factors including the composition, engineering properties, and three-dimensional framework of geologic units underlying a site. The natural and eventual ground moisture profile of a site is an especially important consideration. The risks and hazards posed by expansive soil and bedrock cannot be completely eliminated. However, they can be significantly reduced through proper site-investigation, design, construction, landscaping, and maintenance practices. An awareness of these topics is critical for geologists, engineers, developer, builders, and property owners.

INTRODUCTION

Although many areas in the United States have expansive soil and bedrock, Colorado's semiarid climate and geology combine to make it one of the most severely affected. The volumetric heaving and settling caused by post-construction, swelling and shrinking of clay-bearing soil and bedrock constitutes Colorado’s most costly geologic hazard. Expansive soil has caused varied levels of damage to pavements, driveways and sidewalks; building walls, floors, and foundations; and water and sewer lines in many parts of the state (Figure 1).

The purpose of this paper is to provide an overview of expansive soil and bedrock in Colorado. The discussions, which will focus on the local state of scientific knowledge and professional practice in Colorado, will include the following topics,: (1) clay mineralogy and engineering behavior; (2) occurrence; (3) damage and damage mechanisms; (4) recognition during site and laboratory investigations; (5) mitigative designs and construction methods; (6) site landscaping and maintenance practices; and (7) disclosure of risks to property owners.

Many of the discussions that follow have been modified from Colorado Geological Survey (CGS) Special Publication 43, “A Guide to Swelling Soils for Colorado Homebuyers and Homeowners” (Noe et al., 1997). This booklet, which was created for laypersons having little or no geologic or engineering background, is also the source for many of the figures used herein.

Figure 1. Examples of damage caused by expansive soil and bedrock. Top photos from Noe and others (1997).

EXPANSIVE SOIL AND ITS ENGINEERING BEHAVIOR

Definitions and Terminology

Expansive soil and expansive bedrock contain clay minerals that can attract and absorb water. As a result, these materials swell in volume when wetted and shrink when they dry. Expansive soil contains unlithified clay, while expansive bedrock contains well lithified to weathered claystone and shale (i.e., laminated claystone, per Tourtelot, 1960). Other terms, such as “swelling soil,” “shrink-swell soil,” “heaving bedrock,” and “bentonite” are commonly used to describe expansive clay materials. These materials commonly exhibit ductile (plastic) behavior when moist. In this paper, the general term "expansive soil" will be used to include both soil and bedrock that exhibit swelling behavior. This shorthand terminology is commonly used by the geotechnical community in Colorado and elsewhere.

Clay Mineralogy

Smectite (montmorillonite) is the clay mineral responsible for most expansive soil damage in Colorado. Bentonite is a type of bedrock that is composed of relatively pure smectite. Bentonite layers were originally deposited as volcanic ash, and have subsequently undergone diagenetic alteration. These layers may have especially high swelling characteristics. Other clay minerals, particularly illite and kaolinite, are non-expansive to slightly expansive. Illite-smectite mixtures, having variable relative proportions of the two clay minerals, are common in many of Colorado’s clay deposits. These mixed-mineral assemblages swell to a lesser degree than pure smectite.

The cations that occupy the molecular gaps between individual clay crystals are an important control on swelling behavior. Low-swelling kaolinite and illite have relatively stable interlayer cations. Smectite, however, may contain relatively unstable cations in its molecular gaps, particularly calcium (Ca^{2+}) and sodium (Na^{+}), in proportional mixtures with more stable cations such as magnesium, iron, and potassium. The calcium and sodium cations have the ability to attract and form bonds with water molecules. Sodium smectites are regarded as having the highest free-swelling characteristics (Mielenz and King, 1955).

Engineering Behavior

Swelling and Shrinking – The volume of an expansive soil changes primarily as a result of a moisture change. Swelling and volumetric expansion occurs when moisture is added. Clay minerals may exert a chemical and physical attraction that pulls water molecules into microscopic, interlayer areas between the flat clay plates. In the case of smectite clay, water may be pulled into intercrystalline areas as well. The clay plates are pushed farther apart as more water layers are pulled in (Figure 2). This pushing apart (swelling) of the interlayers can cause high swell pressures and an increase of volume within the mass of soil that is being wetted.

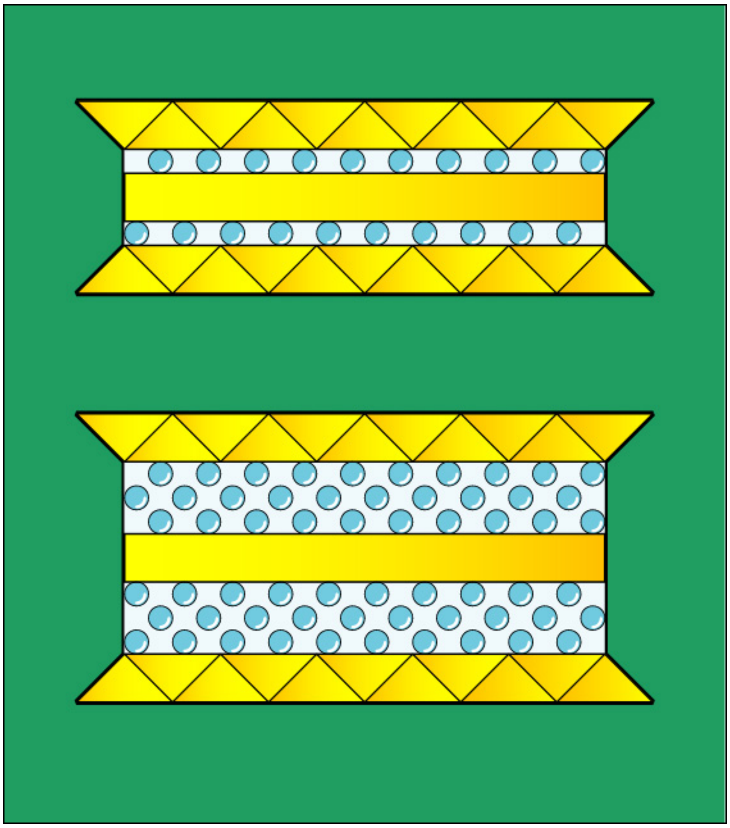

Figure 2. Expanding clay plates, as seen at a microscopic level. Circles indicate water molecules in the interlayer areas. Modified from Hart (1974).

Shrinkage, the opposite effect of swelling, occurs when the soils dry out. As drying occurs, layers of water molecules are pulled out from in-between the clay plates by evaporation or by capillary forces from plant roots. This causes the area between clay plates to collapse on a microscopic level, and may cause a decrease in volume within the mass of soil that is being dried.

Swell Potential – The potential volume expansion of a soil under actual field conditions depends on several factors. A detailed discussion of these factors is beyond the scope of this paper, but a summary is presented below.

1) Type of minerals. See the discussion above, under "Clay Mineralogy." Smectite and mixed illite-smectite are the most common types of clay minerals in expansive soil in Colorado. Soils that are composed of relatively stable clay minerals such as kaolinite or illite, or non-clay minerals such as quartz or feldspar, usually have no to low swell potential.

2) Concentration of expansive clay. The more particles of expansive clay present in a unit volume of soil or bedrock, the greater its swell potential. Material that has a high proportion of kaolinitic or illitic clay, or non-clay minerals, usually has no to low swell potential.

3) Type of interlayer cations. See the discussion above, under "Clay Mineralogy." Research in the Pierre Shale near Denver by Johnson (1998) indicates that the shale contains mixed proportions of sodium, calcium, magnesium, potassium, and iron cations. Calcium is the dominant cation, although sodium dominates in a few samples where gypsum has formed due to pyrite oxidation, depleting the available calcium.

4) Density. A dense material containing expansive clays will have more clay particles and fewer air-filled voids than a loosely packed material of similar mineral composition. The denser material will have a greater swell potential as a result. However, a looser material may expand more quickly because of faster water infiltration.

5) Moisture change. A dry soil has the potential to absorb more moisture than a wet soil, and can subsequently undergo a greater amount of volume expansion. The amount of moisture change that can occur in a soil is a function of the natural moisture content, the ability of the clays in the soil to pull in additional moisture (suction), and the amount of free-draining water or water vapor available to the soil.

6) Overburden pressure. A layer of expansive soil that occurs near the ground surface may swell significantly because there is very little restraining pressure to prevent it from swelling. However, the swell potential of a similar layer that occurs several feet below the surface is reduced by the weight of the surrounding and overlying soil (overburden). If the overburden pressure is greater than the soil's swelling pressure, actual swelling and uplift are unlikely. This is particularly important in the design of foundation loads for structures build on expansive soils.

OCCURRENCE OF EXPANSIVE SOIL IN COLORADO

Expansive soil and bedrock are widespread throughout Colorado. They cover broad areas of the eastern plains, and are found mainly in valleys and on mesa slopes in western Colorado (Figure 3). In the mountainous areas, such soils are limited to valleys between the mountain ranges.

A majority of the state's major population centers are located in areas of potentially expansive soil and bedrock. On a smaller scale, however, individual sites within these areas may not have expansive soil beneath them because of localized geological variations (as illustrated in Figure 4). It is important to realize that the subsurface soil and bedrock may be different than that shown at the surface in geologic maps. In many cases, there may be several different zones of soil and/or bedrock in the subsurface, each having a unique composition, moisture content, and swell potential. These differences necessitate detailed site investigations (Figure 5).

Geologic Units – Numerous bedrock formations and soil deposits in Colorado contain expansive clay minerals. The following descriptions cover the state's major expansive soil-bearing geologic units, by geologic age, from oldest to youngest. The general areal distribution of these units across Colorado may be seen in the general geologic overview paper in this volume (Noe et al., 2003).

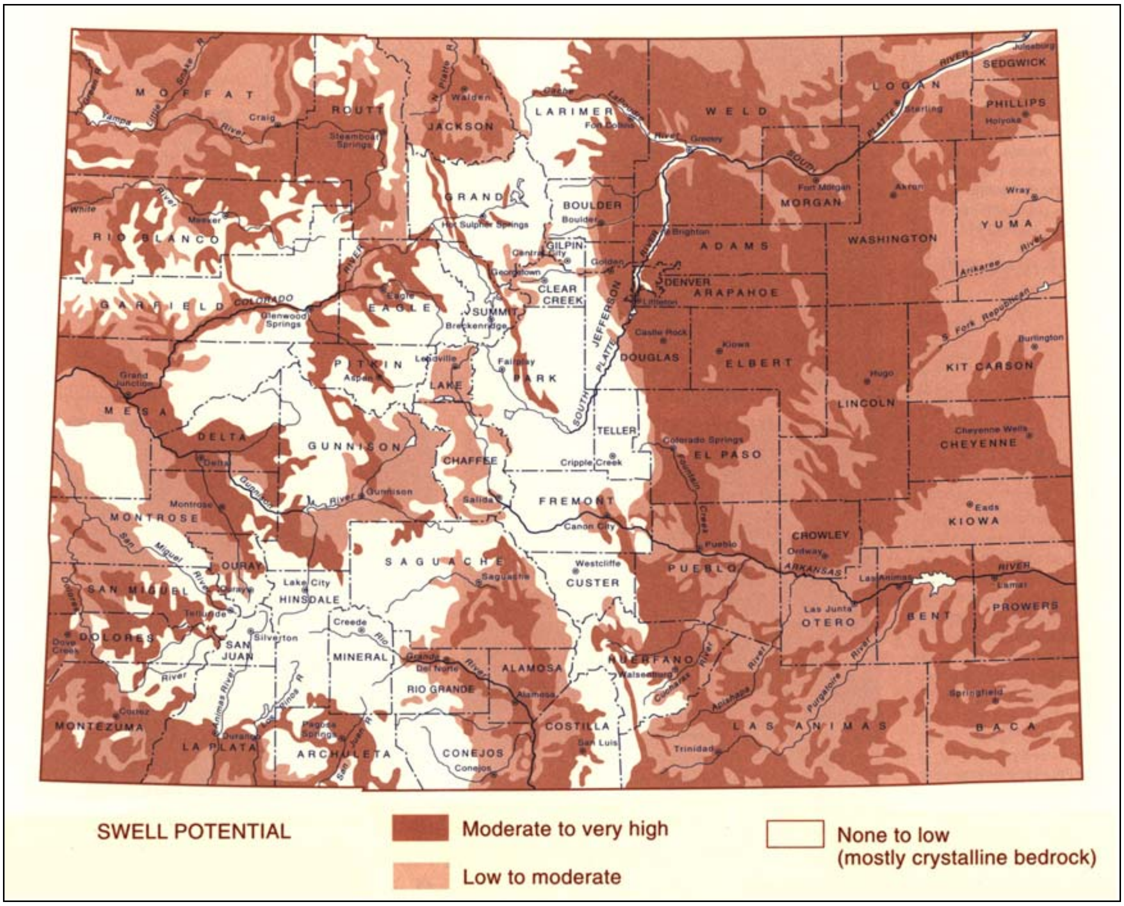

Figure 3. Generalized distribution of expansive soil and bedrock in Colorado. From Noe and others (1997); modified from Colorado Land Use Commission (1974) and Jochim (1987).

Colorado's Precambrian-age metamorphic and granitic rocks are typically non-expansive, although it is possible that some soils derived from the weathering of the granitic rocks could have low swell characteristics. A hyperspectral reflectance, remote-sensing imagery analysis of surficial clay minerals in the Colorado Springs area (Chabrillat et al., 1999) shows that the Pikes Peak Granite, a 1 billion-year-old granitic pluton, is overlain by small areas of kaolinite clay that is assumed to have low plasticity.

The state's Paleozoic-age sedimentary rocks have largely low swell characteristics. Most of the shales associated with these formations are composed of non-expansive silt and low swell, illite or kaolinite clays. A notable exception is the Glen Eyrie Shale Member, in the basal part of the Pennsylvanian Fountain Formation near Colorado Springs. This unit is limited in its areal extent and, although it appears to contain plastic clays, the exact mineralogy is not known.

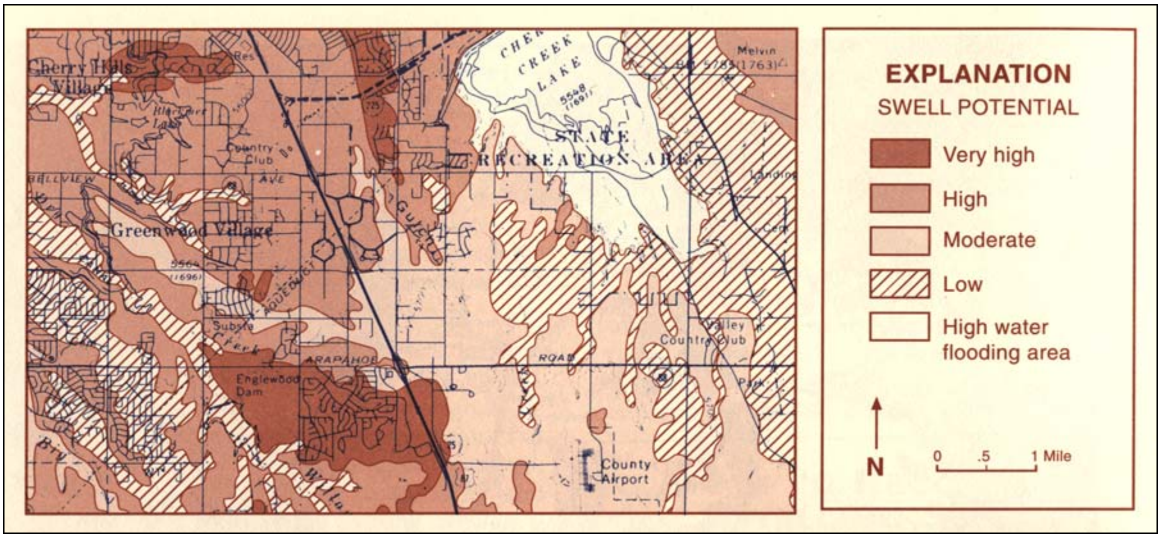

Figure 4. Example of a map showing the local, surficial distribution of expansive soil units. From Noe and others (1997); original map from Hart (1974).

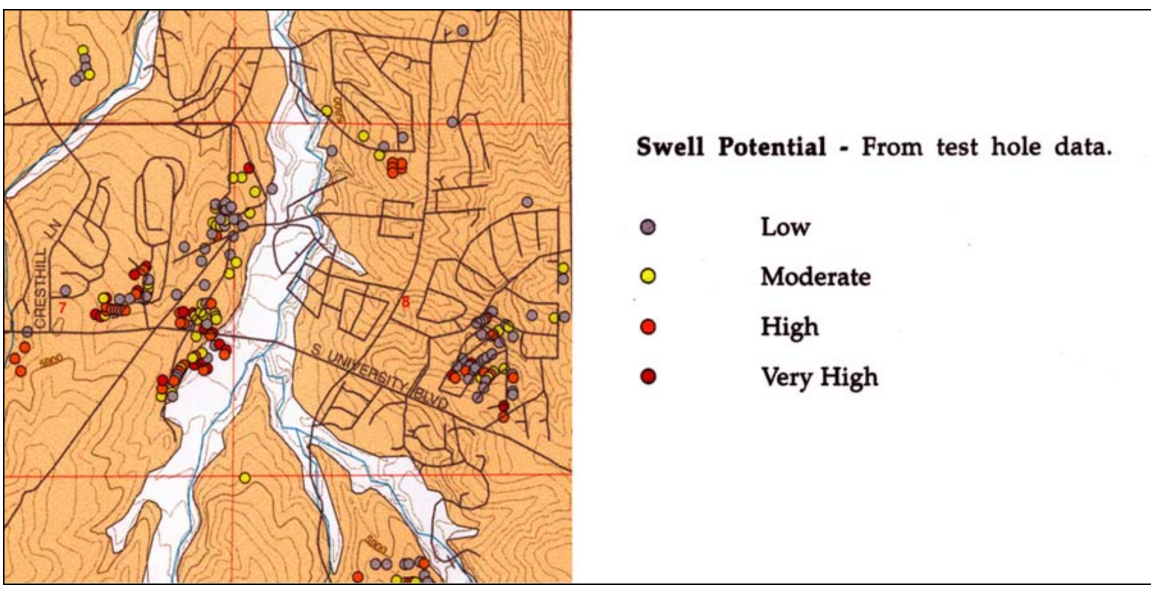

Figure 5. Example of a map showing a surficial area generally mapped as having expansive soil and test hole data that indicates a high range and variation of swell potentials in the subsurface strata. From Berry and others (2002).

The most widespread deposits of expansive bedrock are Mesozoic in age. Notable units include the Jurassic Morrison Formation and the Cretaceous Mancos and Pierre Shales (Figure 6). The Morrison Formation contains non-expansive sandstone and limestone, interbedded with expansive claystone and bentonite, all of which were deposited in a humid, swamp-like setting. The individual beds can be somewhat laterally discontinuous in such settings. The Mancos and Pierre Shales were deposited in a marine setting, in the Cretaceous Western Interior Seaway. These units consist primarily of silty claystone and clayey siltstone, with some sandstone zones and thin (typically less than 1 ft or 0.3 m thick) bentonite beds. The beds are laterally continuous, owing to their marine genesis. The claystone consists of illite-smectite in various relative proportions (Gill et al., 1996; Johnson, 1998; Chabrillat et al., 1999). Although similar in appearance and genesis, the Mancos Shale is siltier and less clayey, and therefore is less expansive in general than the Pierre Shale. This is a result of the location of these marine shales as they were deposited within the facies tract, with the Mancos sediments being deposited closer to the Cretaceous shorelines and closer to the fluvial-deltaic sediment sources in Utah (McGookey et al., 1972).

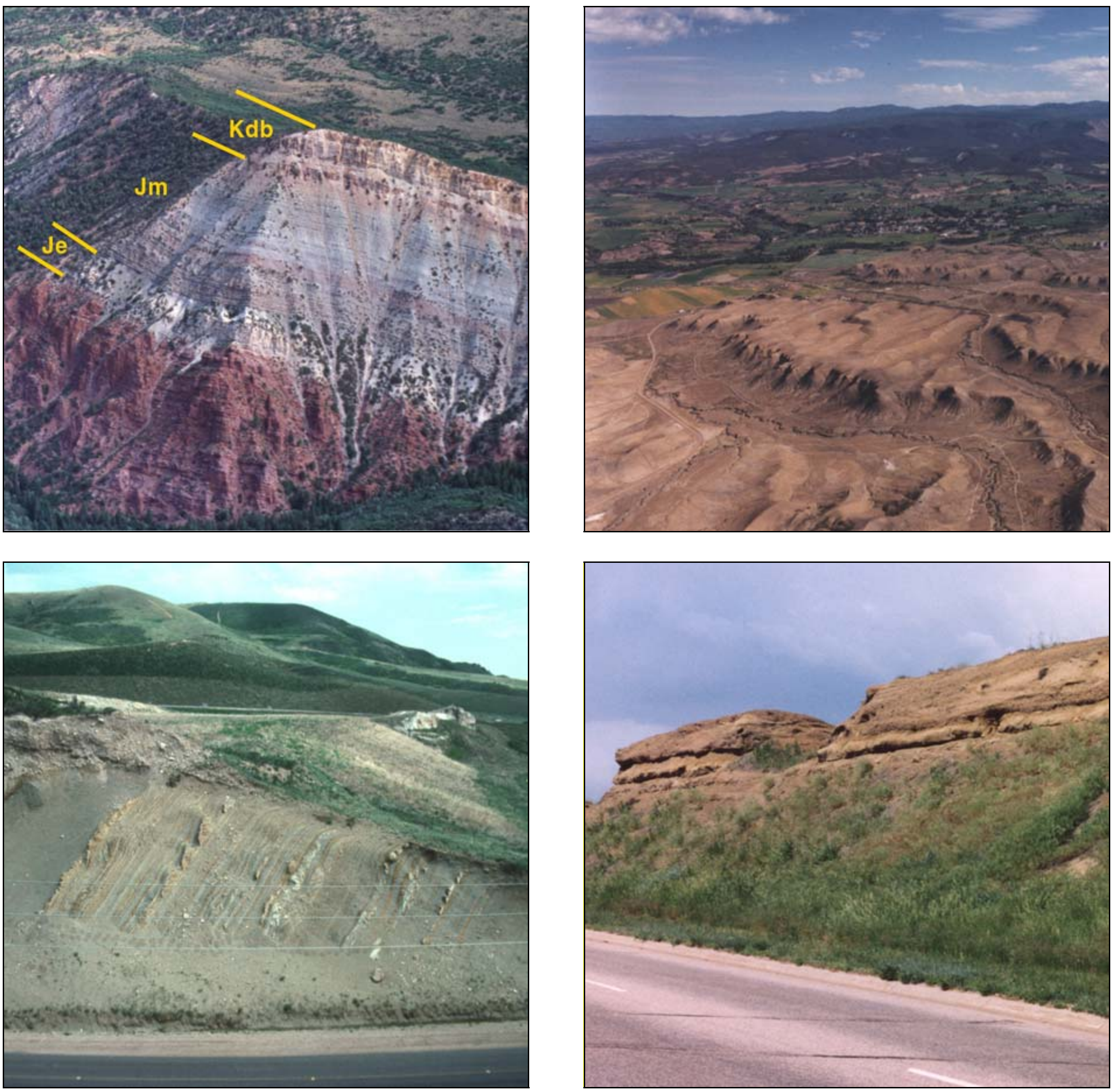

Figure 6. Photos of expansive bedrock formations, clockwise from upper left: Morrison Formation, Mancos Shale, Denver Formation, and Pierre Shale.

Other Cretaceous formations that contain expansive clays include the Graneros (Benton) Shale, Smoky Hill Shale Member of the Niobrara Formation, Laramie Formation, Arapahoe Formation, and the age-equivalent Denver and Dawson Formations in the Denver Basin (Figure 7), where they constitute a widespread expansive bedrock hazard. The Graneros and Smokey Hill Shales are similar to the Pierre Shale in their depositional setting; however, the Smokey Hill Shale is more calcareous and less expansive in general. Both of these units contain thin bentonite beds. The Laramie, Arapahoe, Denver, and Dawson Formations are all continental units that were deposited in alluvial floodplain settings. They contain interbedded sandstone, siltstone, and claystone layers. In particular, the Denver Formation is comprised of andesitic claystone and sandstone derived from volcanic deposits. The claystone can be highly expansive, and even the weathering of volcanic clasts in the sandstone can produce expansive clay soils.

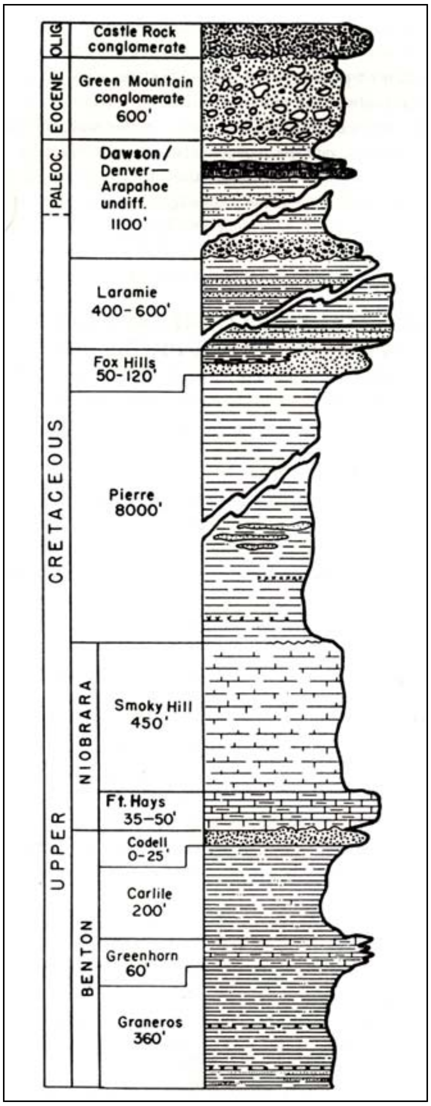

Figure 7. Stratigraphic section of geologic formations in the Denver area. Most of these formations contain expansive claystone beds. From LeRoy (1955).

In western Colorado, the Mancos Shale interfingers with and is overlain by the Mesaverde Group, which contains continental sedimentary rocks (sandstone, siltstone, claystone, and coal) having some discontinuous, potentially expansive claystone beds. The Lewis Shale, which is similar to and partially age-equivalent to the Pierre Shale, overlies the Mesaverde Group and crops out in a narrow strike valley near Durango. Like the Pierre, it contains expansive, silty claystone.

Cenozoic bedrock units in Colorado also contain expansive clay minerals. The previously mentioned Denver and Dawson Formations are Cretaceous to Paleocene (early Tertiary) in age. In the large mountain basins, Tertiary units such as the Middle Park Formation and Troublesome Formation contain expansive clays. Some of these clays also contain volcanic clasts that may weather to expansive clays. The Tertiary San Juan volcanic field is not known for having expansive properties. It is possible, however, that sediments weathered from these volcanic rocks may contain expansive clay minerals.

Many of Colorado's Quaternary-age sediments contain expansive clays. Most often, these clay-bearing deposits are derived from nearby bedrock exposures. Colluvial and residual soil deposits are often clayey and expansive, especially where they overlie clay-rich Cretaceous and Tertiary bedrock. Other types of soil deposits that may contain transported expansive clays include alluvial, landslide, debris flow, glacial till, and moraine deposits. In addition, loess (windblown silt) deposits cover large areas of Colorado. Some of these deposits contain silt-sized aggregations of clay particles. These loosely packed deposits can undergo expansive-soil swelling and ground heaving, in addition to collapse of void spaces and ground settlement when wetted.

Steeply Dipping Bedrock – Expansive, steeply dipping bedrock (also known as heaving bedrock) constitutes a distinct geological hazard in certain areas of Colorado near the base of mountains where the sedimentary bedrock layers are upturned and tilted (Figure 8). In such areas, the bedrock layers may swell unevenly to form parallel, linear heave features along the ground surface (Figure 9). Houses built over such heave features may be subjected to extreme amounts of vertical and lateral stress, and the resulting damage can be severe.

The mechanisms responsible for heaving bedrock movements are geologically complex. Heaving may occur due to uneven swelling of individual bedrock layers, each having a different swell potential, or due to shear-slip movements along bedding planes or fracture surfaces (Figure 10). The processes that cause heaving bedrock are not well known. Rebound (expansion of the clay minerals as a result of sudden unloading) may be a factor, in addition to water-induced swelling of clay particles in the bedrock. Moisture can penetrate a greater depth into steeply dipping bedrock than in flat-lying bedrock, resulting in a deeper zone of potential swelling.

Many construction designs commonly used to mitigate the impacts of expansive soils have met with limited success in areas of differentially heaving bedrock. For example, drilled pier foundations have been damaged in numerous cases. The basic assumption for those designs is that the bedrock is stable. This is not the case for heaving bedrock, because certain strata that make up the bedrock are swelling and deforming. One method that may counteract the differential heaving is overexcavation and fill replacement (also called deep sub-excavation), whereby a house is isolated from the bedrock by a thick pad of engineered fill.

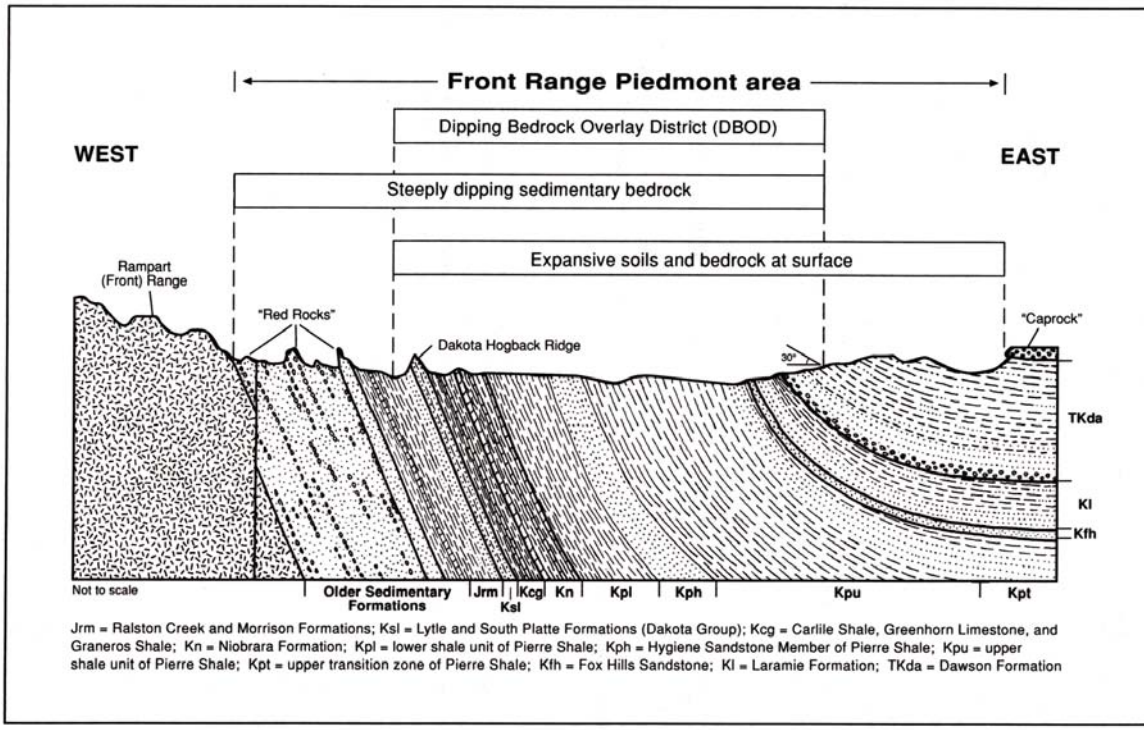

Figure 8. Schematic cross-section of geologic formations in northwestern Douglas County, showing the overlap of expansive and steeply dipping bedrock zones. From Noe and Dodson (1999).

Figure 9. This "roller-coaster road" in Jefferson County, near Denver, is the result of uneven swelling and heaving of steeply dipping bedrock layers in the Pierre Shale. From Noe (1997).

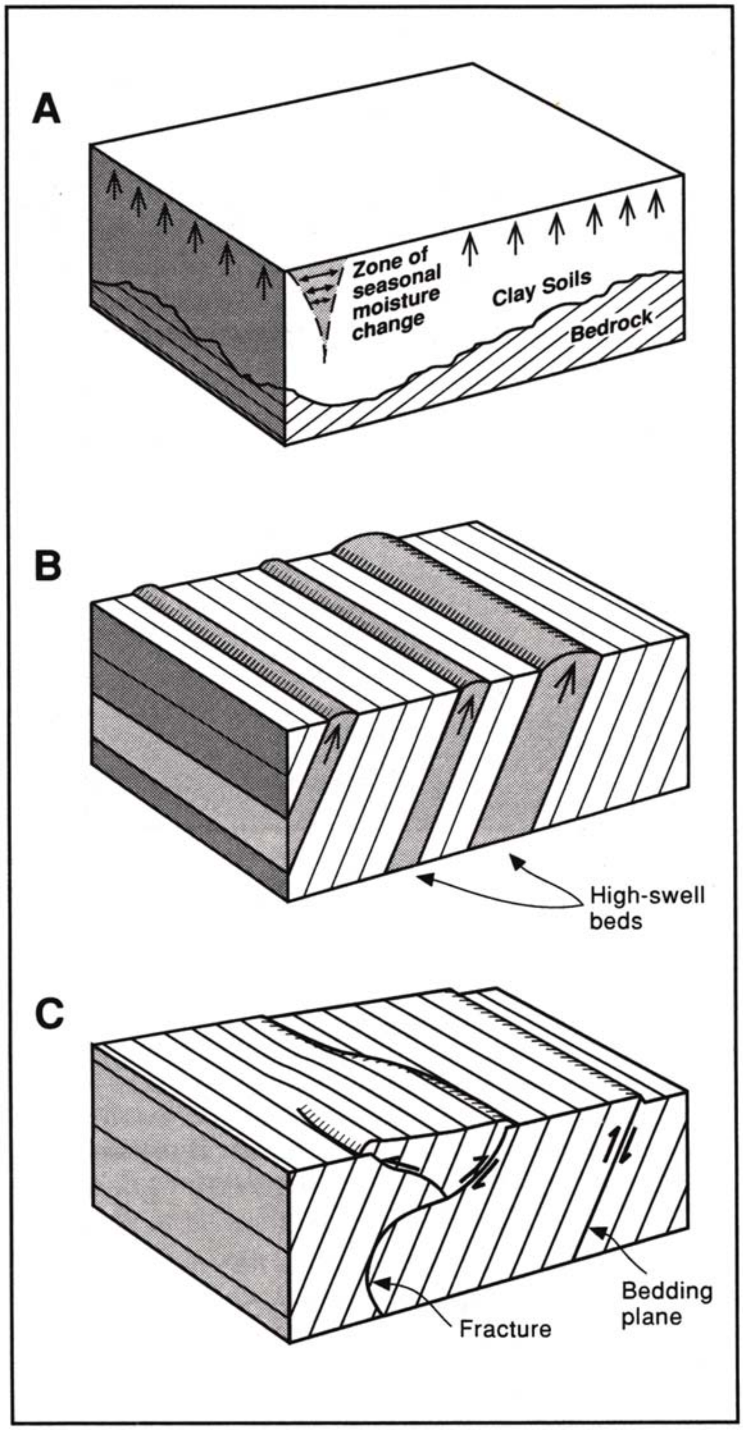

Figure 10. Different types of expansive soil and heaving bedrock. (A) General model for expansive soil, showing vertical heaving within the active zone of moisture change. (B) Symmetrical heave features caused by uneven swelling of bedrock layers. (C) Asymmetrical heave features caused by shear-slip movement along bedding planes or fracture surfaces. Image from Noe and Dodson (1999).

Jefferson and Douglas counties now require more detailed site investigation and specialized building techniques where heaving bedrock conditions exist. These areas are identified in overlay maps that show the extent of potentially heaving bedrock. Houses in the overlay areas constructed before 1995 may not have been built with current state-of-the-art construction practices. Similar geological conditions where heaving bedrock may occur exist at other locations along the Front Range foothills, and on the Western Slope of Colorado. The author has observed differential heaving of roads and buildings in areas of steeply dipping, near-surface bedrock in Colorado Springs and New Castle. For a more complete history of Colorado's dipping bedrock problem and its solutions, see Noe (1997). In addition, two papers in this volume describe the trenching approach to site exploration in areas of dipping bedrock (Noe, 2003a) and a history of the deep sub-excavation approach and a related case history (McOmber and Glater, 2003).

Climatic Controls – Figure 11 shows the average annual water balance in Colorado, based on the combined effects of precipitation and evapotranspiration. The mountainous areas of Colorado have relatively high rates of precipitation and a cool climate, and correspondingly low rates of evapotranspiration. In these areas, there is usually a surplus of ground moisture. High-plasticity soils in the higher mountain valleys may have elevated moisture contents, especially within poorly drained and formerly glaciated valley bottoms. These moist to wet soils do not have high swell potentials in their natural state.

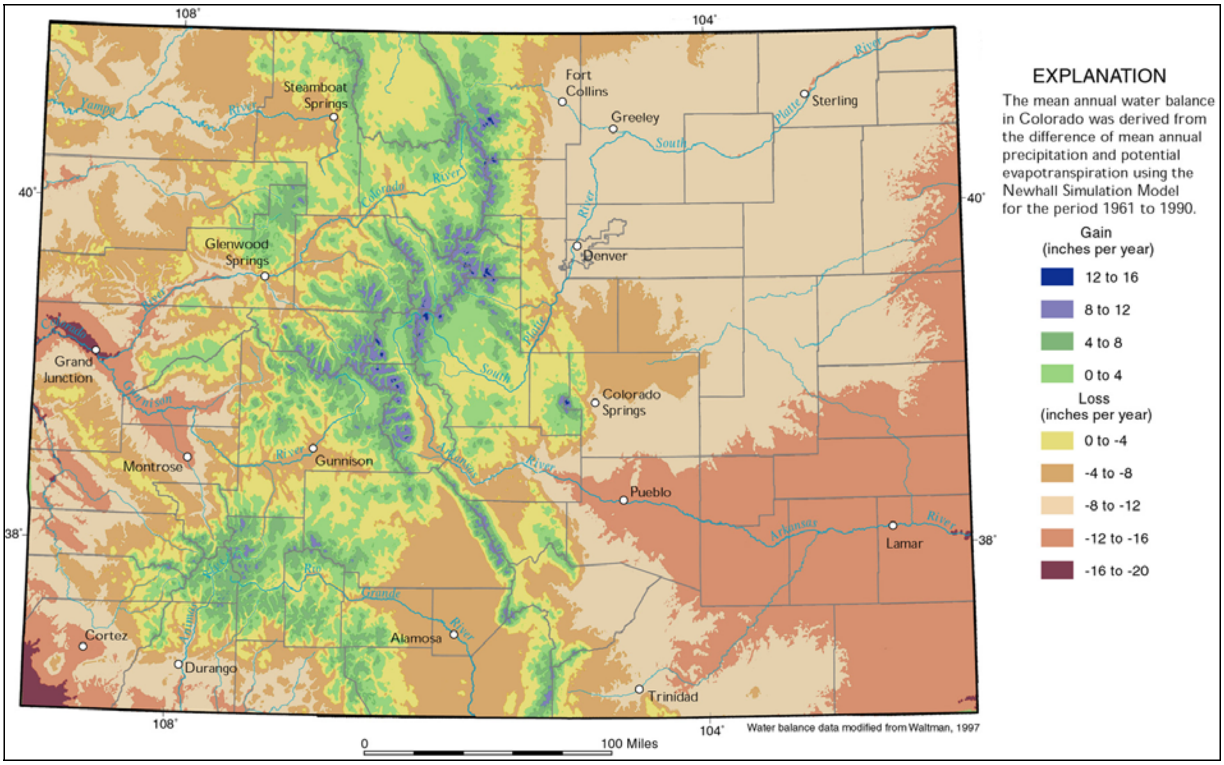

Figure 11. Average annual water balance in Colorado. From Topper and others (2003); modified from Waltman (1997).

The eastern plains and western valleys, where most of Colorado's expansive soils are found, receive less precipitation than the mountains. The major population centers receive on the order of 8 to 16 in (20 to 41 cm) of precipitation per year, on average (U.S. Department of Agriculture and Natural Resources Conservation Services, 1999). These areas have hot summers, cold winters with large temperature fluctuations. They also have high rates of potential evapotranspiration, on the order of 30 to 40 in (76 to 102 cm) per year, on average (Farnsworth et al., 1982). As a result, most of the populated, non-mountainous areas of the state are characterized by an overall deficit of water during much of the year, and the near-surface soils are typically dry. Accordingly, many of the state's clay-rich soils may have high swell potentials in their natural state.

DAMAGE AND DAMAGE MECHANISMS

Magnitude of Damage

Expansive soil causes significant damage and is a threat to property and facilities in many parts of the world. Various estimates have been made of the magnitude of damage. Jones and Holtz (1973) conducted a nationwide study, on the behalf of the ASCE Research Council on the Expansive Behavior of Earth Materials, to estimate the cost of repairing damage caused by expansive soil movements. Their estimate, 2.3 billion dollars yearly, exceeded the cost of damage from other natural hazards by a factor of two. This estimate was made before some of the "super disasters" of the 1990s, including Hurricane Andrew (1992), the Mississippi River flood (1993), and the Northridge earthquake (1994). It appears that there are several types of natural disasters that could rival or exceed expansive soil in magnitude for a particular year or event, but the long-term, average annual cost of damage is probably highest for expansive soil hazards.)

In Colorado, Holtz and Hart (1978) estimated that expansive soil causes about 16 million dollars worth of damage to public facilities alone. Chen (1988) estimated that one of ten houses in the Denver metropolitan area was built on expansive soil or bedrock. He predicted that one of three houses built on highly expansive ground would be adversely affected. Costa and Bilodeau (1982) found that 50% of the greater Denver area is underlain by moderately expansive soil and rock at the ground surface, while another 25% is underlain by highly expansive soil and rock.

There has been no recent, comprehensive estimate of damage from expansive soil in Colorado. This is due, in part, to the "non disaster" nature of the damage (i.e., because it is not sudden and episodic, expansive soil damage is not covered by FEMA programs, and therefore the losses are not centrally reported or tallied). Also, because the damage may occur slowly over time, individual buildings or facilities in an area may be affected at different times.

Losses from expansive soil damage in the state are borne by a wide variety of entities. Home-warranty insurers have historically incurred large losses because of numerous, high-payout claims in Colorado. Likewise, lawsuits have been numerous over the years, most of which have been settled out of court. In early 1996, a precedent-setting court decision awarded 533 million dollars to 957 homeowners in the suburban Denver area who sued a homebuilder in a class-action lawsuit over floor designs used for expansive soil. This was followed shortly by two settlements between homebuilders and 253 and 12,300 homeowners in other, similar class-action lawsuits (with settlements of 2.6 million dollars and an unspecified amount that may eventually be as large as several tens of millions of dollars, respectively). These lawsuits and settlements, which were closely followed and widely reported in the Denver newspapers, represent some of the largest homeowner settlements in the nation.

Owners of older homes (i.e., those homes that are beyond their warranty periods) are generally solely responsible for the cost of repairing damages. In many cases, damage to older homes is not repaired or reported because the owner cannot afford to make repairs or chooses not to make the repairs. Different performance expectations by homeowners or facility owners may result in an inconsistent reporting of damage.

The Role of Subsurface Moisture

The Active Zone – The presence of and changes in subsurface moisture can cause serious problems for a facility built on or in potentially expansive soil. The amount and distribution of subsurface water within the soil profile may vary seasonally under natural conditions. In most areas of Colorado, the amount increases during the late winter and spring, when rates of natural infiltration from precipitation are high, and during the summer in areas where there is artificial irrigation. During the dry season of fall and early winter, it may decrease again. Similarly, the underground water table and the zone of capillary saturation may rise during the wet periods and fall during the dry periods. The depth below the ground surface where soils undergo seasonal wetting-drying cycles is called the active zone or zone of moisture change (Figure 12).

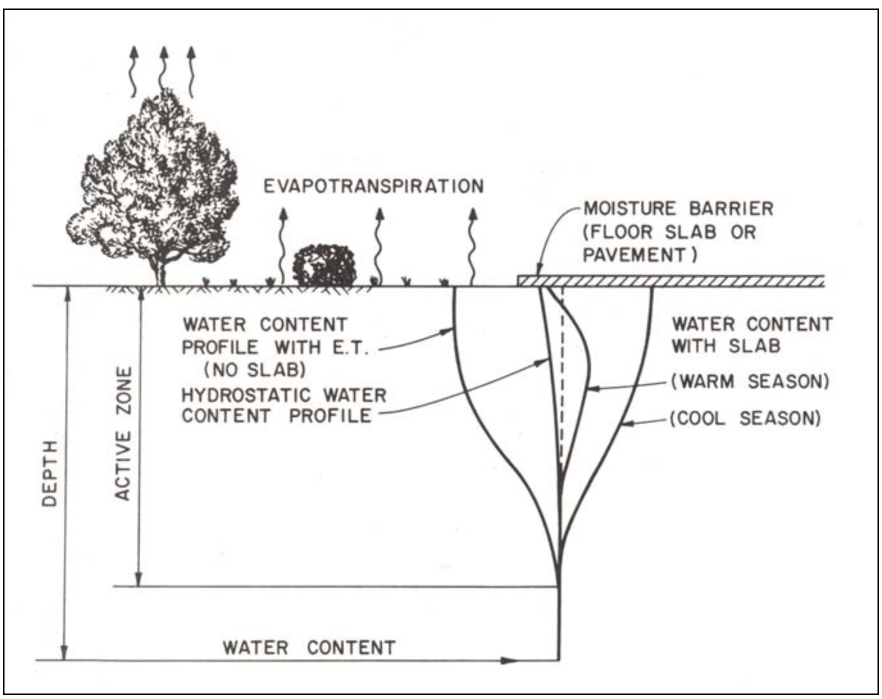

Figure 12. Soil profile showing the near-surface zone of moisture change, called the active zone. From Nelson and Miller (1992).

Under natural conditions, seasonal wetting and drying cycles cause expansive soils to swell and shrink to some extent. This is not a problem if the land is being used for agriculture or is undeveloped. However, building a subdivision in an area can significantly alter the natural moisture content of the soil over time. Water infiltration increases due to irrigation of lawns and gardens and, in some cases, leakage from septic systems and water or sewer pipes. At the same time, impervious roadways, parking lots, driveways, sidewalks, and buildings footprints reduce evaporation and may concentrate runoff into other areas. Off-site water may migrate into an area through backfill trenches and gravel bedding. Perched water tables may develop. The overall result is a net increase in soil moisture.

The natural active zone along Colorado's eastern plains is typically 7 to 10 ft (2.1 to 3.0 m) deep. The post-construction zone of wetting in this area typically increases to depths of 10 to 15 ft (3.0 to 4.6 m). In areas of steeply dipping bedrock, the zone of wetting may increase to depths of 35 ft (10.7 m) after a subdivision is built. The topic of water infiltration through time and its affect of the soil profile is of great interest to the Denver geotechnical industry, as it has been a point of contention in recent lawsuits. In particular, there is disagreement over whether a wetting front can move infinitely downward in a soil through time. The Colorado Association of Geotechnical Engineers (CAGE) is currently conducting an area-wide study of soil-moisture profiles in the Denver metropolitan area. For a recent case study involving transient infiltration and wetting of expansive soil, see the paper in this volume by Vessely and others (2003).

Swelling Versus Shrinking Behavior

Swelling of clay soil and bedrock, and associated heaving, accounts for most of the damage to structures, roads, and facilities in Colorado. Colorado soils are usually dry in their natural condition, but tend to become wetter after subdivisions are constructed and occupied because additional sources of water become available. In addition to natural sources of subsurface water (e.g., rainfall, snowmelt), other significant sources may come from human activities such as lawn and crop irrigation, concentrated runoff from roof gutters and impervious areas, and seepage from man-made ponds and ditches and buried water and sewer lines. Perched water tables, formed when infiltrating moisture encounters a low-permeability horizon and spreads laterally within the overlying layer, may contribute locally to the overall swelling and heaving.

Colorado is subject to occasional periods of drought. Many Colorado municipalities limit lawn and garden irrigation during such times. During a drought, evapotranspiration will exceed water infiltration, and the active zone will dry out. If expansive soils are present in the active zone, they will undergo volume shrinkage. This may reverse the direction of heaving and reduce the amount of offset that has occurred during earlier periods of swelling, or it may cause additional damage due to near-surface settlement of the soil.

During Colorado’s most recent drought (2001-2003), the author is aware of numerous cases of soil-shrinkage-related damage to foundations and utility connections; in some cases, damage occurred in both younger and older neighborhoods following the implementation of lawn-watering restrictions. This illustrates that expansive clay soils will react to dynamic changes in soil moisture, to wetting or drying episodes, no matter how much time has passed since the previous change in moisture.

Certain types of trees and plants will pull large amounts of moisture out of the soil during drought periods. This may cause localized shrinkage and settling of the ground surface in the immediate area of the tree. Damage to structures may occur if the tree is located close to a house foundation.

Oxidation of Pyrite

The oxidation of pyrite in claystone is recognized as a cause of volumetric expansion, heaving, and damage in some parts of the eastern U.S.A. and Canada (e.g., Penner et al., 1973). This occurs as the claystone is unearthed and is exposed to the atmosphere. As the rock weathers, pyrite minerals in the claystone are chemically altered to form gypsum and iron oxides, such as jarosite. The gypsum, which is a hydrated calcium sulfate mineral, expands the rock through the process of crystal growth. Many of Colorado's marine claystone formations contain pyrite, and several formations (e.g., Mancos and Pierre Shales) contain numerous gypsum-filled veins in weathered exposures.

To the author's knowledge, there have been no studies of this phenomenon done in Colorado or on Cretaceous shales from the Western Interior. However, expansive sulfate growth in lime-treated expansive soil and bedrock has been an issue at Denver International Airport and in other areas (Little and Petry, 1992; Burkart et al., 1999). The specific association of and contribution of pyrite oxidation to swelling in Colorado is, as of yet, unreported and unknown.

Rebound

The ability of a plastic material to expand to regain its former shape after being loaded is called rebound. In geologic settings, rebound is associated with rock that is overconsolidated (i.e., previously loaded by sediments or ice and then unloaded again). Many of Colorado's claystone formations are overconsolidated. In theory, near-surface claystone seams could undergo some amount of rebound-related expansion if the overburden was suddenly removed. Some researchers in Colorado have attributed the heaving of steeply dipping bedrock to rebound (e.g., Nichols, 1992). However, in the author's experience, it appears from numerous observations that sudden wetting of new bedrock exposures is the cause of most of the episodic heaving in this setting.

RECOGNITION DURING INVESTIGATIONS

For facility design purposes, it is important to recognize the presence of expansive soil layers, and to assess their extent in the subsurface and the potential magnitude of swelling. The presence and extent of such deposits are determined by field investigations, while the swell characteristics are determined by laboratory testing of samples.

Site Investigations

The purpose of an engineering geologic site investigation is to identify geologic conditions that constitute constraints or hazards to development. In the case of expansive soil, this entails recognizing the presence and three-dimensional geometry of such deposits at the surface and in the subsurface.

The presence of expansive clays at the ground surface can be confirmed by observation. Soils containing expansive clays will be very sticky when wet, and very hard when dry. Desiccation cracks or a puffy "popcorn" texture (Figure 13) typically form when near-surface expansive soil dries out. Such features may not be evident where topsoil or heavy vegetation covers the native soil. Large desiccation cracks, with depths of up to several feet (up to a meter or more), may form during extended dry periods. These larger cracks play an important role in expansive soil behavior, as they allow for deeper evaporation and shrinkage during dry periods and deep penetration of water during subsequent wet periods.

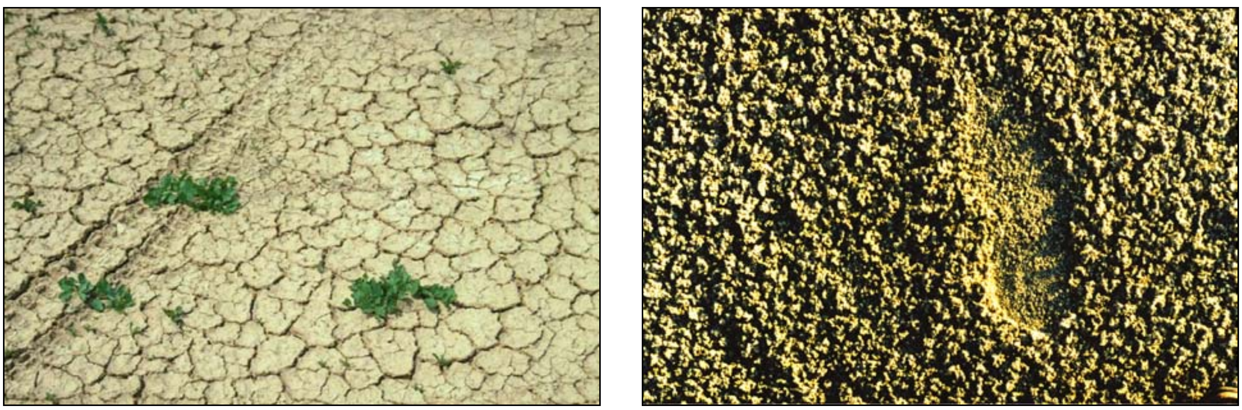

Figure 13. Evidence of expansive soils at the ground surface. (A) Small-scale desiccation cracks in soil containing expansive clay; note tire tracks for scale. (B) "Popcorn" texture in soil containing expansive clay having very high swell potential; note footprint for scale. From Noe and others (1997).

Besides surface evidence, it is important to identify whether deeper layers or lenses of expansive soil are present beneath a property. The evaluation of subsurface layers is most often done by drilling one or several test holes or by digging a trench. Drilling is effective for relatively flat-lying soil and bedrock because it allows for inspection and sampling of successively deeper layers. Trenching is more effective in areas underlain by steeply dipping bedrock because it exposes many near-surface bedrock layers for inspection and sampling. For a more detailed description of the trenching process, as applied to in-field characterization of steeply dipping bedrock, see the paper in this volume by Noe (2003a).

Samples are recovered for laboratory testing from test holes, using rig-driven California samplers or Shelby tubes, or from trenches, using grab samples or hand-driven brass tubes. There are no statewide specifications in Colorado for test hole spacing or sample density. Certain local government regulations (e.g., the Jefferson County Subdivision Regulation) and professional organization documents (e.g., Colorado Association of Geotechnical Engineers, 1996) contain minimum spacing and sampling recommendations and standards for use in site investigations.

Laboratory Investigations

Samples taken from test holes or trenches are tested for a variety of engineering properties in a laboratory. This information is used to design foundations for buildings. Such evaluations are a required practice in many areas of Colorado where expansive soil is anticipated. These tests commonly include natural moisture content (ASTM D 2216), natural dry density (ASTM D 2937), particle size passing #200 sieve (ASTM D 0422), and Atterberg limits (ASTM D 4318). Of these, the Atterberg limits – liquid limit, plastic limit, and plasticity index – are used as a means of assessing the plasticity of a sample. A plot of liquid limit vs. plasticity index (i.e., a Casagrande chart) indicates whether a material is classified as a clay or silt, and whether it is of high or low plasticity, under the Unified Soil Classification System (ASTM D 2487).

The comparison of these limits with the water content gives an indication of the field condition of the sample (i.e., whether it is relatively dry and in a semi-solid state, or relatively moist and in a plastic state – this is a rough indicator of how much moisture a clay sample can take on.). Particle size testing, in common practice in Colorado, involves sieving to discern the total fraction (i.e., the –200-sieve fraction) of clay and silt particles. The hydrometer method of particle size testing (ASTM D 1140), which yields the relative percentage of clay vs. silt, is typically not used because it is too time intensive, and because pier-foundation design methods typically do not incorporate clay percentage data.

Swell-consolidation testing, also known as the Denver Swell Test (ASTM D 4546) is used to compute a sample's swell potential and swelling pressure. These are two measurements of a soil's ability to expand against different restraining pressures under laboratory conditions. The tests are typically run under surcharge loads of 300 psf (for roads) or 500 psf, 1,000 psf, or at loads that approximate the final, anticipated overburden pressure (for structure foundations). Soils are rated as having very high, high, moderate, low, or no swell potential (Table 1). Swelling pressure is the pressure exerted by the soil mass against a restraining force when it is wetted. Typical swelling pressures for expansive soil in Colorado can exceed 15,000 psf, with some values reaching or exceeding 30,000 psf. Soils having such high swelling pressure are capable of causing uplift to concrete slabs and footing-type foundations, which exert relatively low loading pressures.

Table 1. Relationship between percent swell, surcharge load, and swell potential rating from swell-consolidation tests. From Jefferson County Expansive Soils Task Force (1994) and CAGE (1996).

Test Surcharge Pressure	**Swell Potential Rating**			
	Low	Moderate	High	Very High
	Swell (% Total Volume Change)			
500 psf	0-3	3-5	5-8	>8
1,000 psf	0-2	2-4	4-6	>6

More recently, some geotechnical consultants in the Denver area have begun to use filter-paper suction testing (ASTM D-5298) and other types of suction testing as a means of estimating the swell potential and potential swell of a soil sample. For a related case study of modeling potential heave using suction-test results, see the paper in this volume by McOmber and Glater (2003).

Caution must be taken when interpreting the results of laboratory tests. According to Jones (1972),

> *The biggest shortcoming of laboratory test information is that it identifies characteristics of soils that have been remoulded (sic) and then exposed to some unnatural environmental extremes. A soil's potential volume change thus defined in the laboratory may bear little resemblance to its actual behavior in situ under natural conditions or natural conditions as modified by man.*

Remedial Investigations

The evaluation protocol for remedial investigations in an area of expansive soil damage is similar to that of undeveloped sites. However, additional emphasis is placed upon determining the in-situ changes that have taken place since development (e.g., to the moisture and suction profiles) and estimating the remaining amount of potential heave (Thompson, 1997). In cases where differential movement has occurred, it is important to establish whether the movement is due to localized expansion in one place, or localized settlement in another, or both, as this may determine the remedial design needs. An example of a school building having damage that was first attributed to settlement and later evaluated as heave is given in Noe (2003b). In such cases, a full assessment of the subsurface geology and the engineering properties of different units is necessary.

MITIGATIVE DESIGNS AND CONSTRUCTION METHODS

Facilities construction on expansive soil involves many types of specialized designs and construction methods. Many variations of a design are possible, depending on the condition of the underlying soil. Critical considerations for a project include site preparation and grading, and the design and construction of building foundations, floors, interior walls and piping, utilities, and subsurface and surface drainage systems. Quality control of construction is crucial for each step of the construction process. This section outlines common considerations used in practice in Colorado; a detailed discussion of these designs and methods is beyond the scope of this paper. For more on these topics, see Chen (1988), Greenfield and Shen (1992), Nelson and Miller (1992), and Noe and others (1997).

Ground Preparation and Grading

Before any houses can be built in a new subdivision, the site is usually graded and shaped, and utilities and roads are installed. This may involve cutting away topographically high areas such as hills and filling in lower areas. Expansive soils or bedrock may be exposed or brought nearer to the surface in grading cuts, and they may make up a sizable portion of the materials used to construct fill pads for houses and roads. There are several methods of site preparation available to reduce the potential swelling of fills and natural soils.

Fills – It is common engineering practice to reduce the swelling potential of graded fills by controlling their moisture and density. The final moisture content of an engineered fill is almost always greater than for most Colorado soils in their natural condition. As a result, the fills may be less prone to swell. Construction of engineered fills may result in mixing of non-swelling materials such as sand or low-swell clays with higher-swell clays, which may effectively reduce the overall concentration of expansive clays.

Cuts – Cut areas exposed by grading are susceptible to swelling because the natural restraining loads have been removed, exposing soil or bedrock layers that have not previously swelled to their full potential. Such areas can dry out to some depth after grading, thereby increasing the swell potential. In some cases, grading exposes fractures or other water conduits that were not open to moisture intrusion prior to grading. Rebound heaving is an additional consideration.

Deep Sub-excavation – Also called overexcavation, this method of cutting and fill replacement is sometimes used in areas of highly expansive soils and bedrock. Overexcavation involves cutting and removing the rock or soil to a prescribed depth, usually 3 to 10 ft (0.9 to 3.0 m) below the anticipated lowest-floor or road level. The cut is then fully or partially filled with uniform layers of original or imported soil under controlled moisture and density conditions. This fill creates a buffer between the foundation or road and the underlying expansive soils. Overexcavations and deep fills may be recommended in certain Colorado counties where steeply dipping, heaving bedrock is encountered. For a more detailed description of this method and a case study, see the paper in this volume by McOmber and Glater (2003).

Chemical Treatments – Another means of reducing swell potential is to mix or inject chemicals into the soil during site grading. Chemical treatments, which are formulated to change the clay chemistry and mineralogy so that the clays become less expansive, are used mainly for roads and larger commercial building sites. A main drawback is that the treatments may not penetrate very deeply or uniformly into most expansive soil and bedrock due to the presence of fractures, low-permeability layers, and other geological complexities. Another potential drawback is that the chemicals may be leached out of the soils over time.

Building Foundation Designs

Building foundations must be properly engineered to account for geological conditions at any given site. Depending on the swell potential, expansive soils may or may not be a primary consideration. Several different types of foundations are commonly used in areas of expansive soils in Colorado. The actual choice of foundation type depends on numerous geologic and non-geologic factors, and may reflect common regional practices and individual preferences of foundation engineers.

Spread Footing – This type of foundation consists of continuous concrete strips, upon which a foundation wall is built (Figure 14A). This design places a relatively large bearing area in contact with the ground, which spreads out rather than concentrates the weight of the house. It works best in loose, non-swelling soil to reduce settlement. Spread footing foundations are generally not recommended where moderately to highly expansive soil is encountered, unless they are used as part of an overexcavation and fill replacement design.

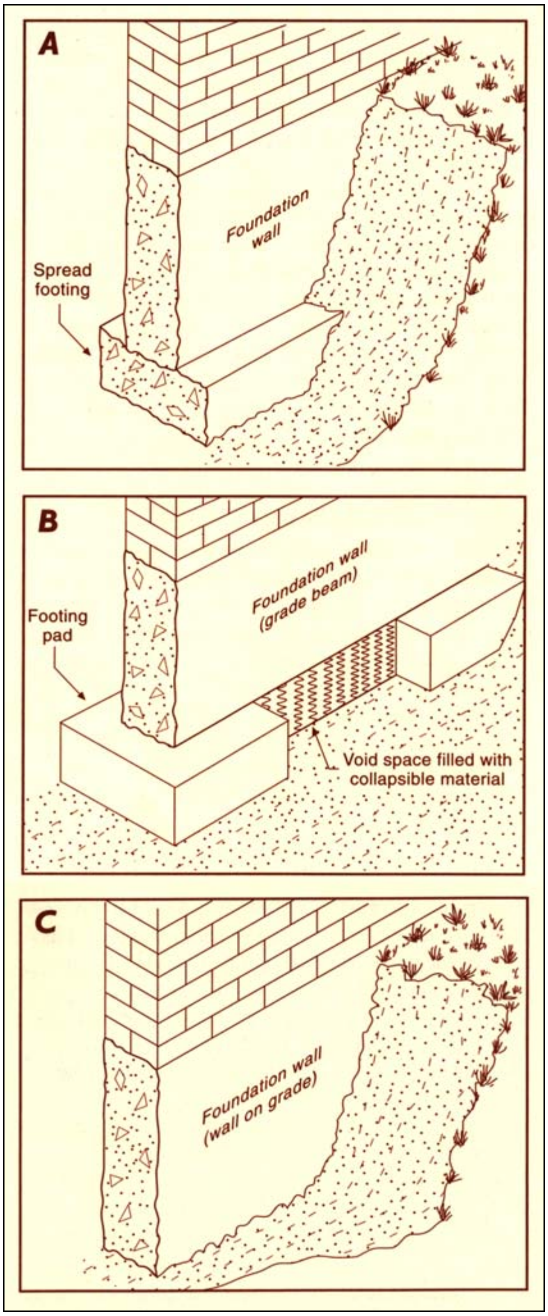

Figure 14. Shallow footing-and-wall type foundation systems. (A) Spread footing. (B) Discontinuous footing pad. (C) Wall-on-grade. From Noe and others (1997); modified from Holtz and Hart (1978).

Footing Pad – This type of foundation consists of discontinuous concrete pads that are spaced apart at specified intervals (Figure 14B). Between the pads are void spaces filled with a collapsible material. A grade beam spans the pads and void spaces. The grade beam and pads support the load of the building. This type of foundation may be appropriate for soil having a very low to moderate swell potential.

Wall-on-Grade – A wall-on-grade foundation consists of a continuous foundation wall that rests directly on the soil (Figure 14C). The wall exerts a moderate pressure on the soil due to its rather small bearing area. This type of foundation has been used in Colorado for soils having low to moderate swell potentials. A variation of the basic design, a voided wall-on-grade foundation, allows for a smaller bearing area, concentrating the load on the underlying soil. This type of foundation has been used in Colorado for soils having moderate to high swelling pressures. However, in recent years it has been largely supplanted for new construction by drilled piers.

Post-Tensioned Slab – This type foundation consists of a concrete slab that has waffle-like beams along the lower side and is smooth on the upper side, forming a mat or raft. Strong steel cables, or tendons, cross through the slab. These tendons are tightened (tensioned) at intervals of time after the concrete is poured, so that the slab becomes stronger and more rigid as the concrete cures. The load-bearing walls of the building rest on the upper surface of the slab. Post-tensioned slabs have relatively large bearing areas and may be uplifted by moderately to highly expansive soil; however, the rigidity of the slab may allow the building to move as a unit to reduce damage. This type of foundation is most often used in Colorado for commercial or multi-family buildings that have large floor areas. It is rarely used for residential buildings with basements.

Drilled Pier – Drilled pier foundations (Figure 15) are the deep foundation systems most often used in areas of moderately to very highly expansive soil in Colorado. Drilled piers for houses are typically constructed by drilling a series of specifically positioned, 8- to 16-in (20- to 41-cm) diameter holes, which are filled with concrete and reinforced with steel rebar. A grade beam is constructed over the piers to create a load-bearing span between them. Void spaces, filled with

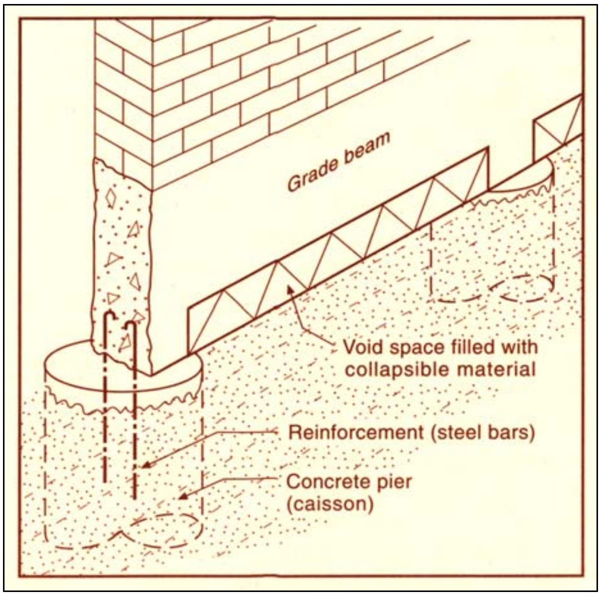

Figure. 15. Drilled pier foundation. From Noe and others (1997); modified from Holtz and Hart (1978).

collapsible material such as corrugated cardboard, are created between the piers to separate the top of the soil from the bottom of the grade beam. Drilled piers typically range between 10 and

30 ft (3 to 9 m) in length from top to bottom, depending on the soil and subsurface moisture conditions.

Drilled pier designs allow the load of a building to be concentrated on a relatively small number of piers. This allows the piers to resist uplift pressures from expansive soil. The piers must be drilled to a certain minimum depth below the zone of expected post-construction moisture penetration (Figure 16), or else they may heave upward and damage the building. Drilled pier foundations may reduce the effects of expansive soil when designed and constructed properly. There are certain geological situations in Colorado, however, where drilled piers may not be the most appropriate foundation design. This includes areas of steeply dipping bedrock, where the bedrock may be unstable to depths of more than 30 ft (9 m).

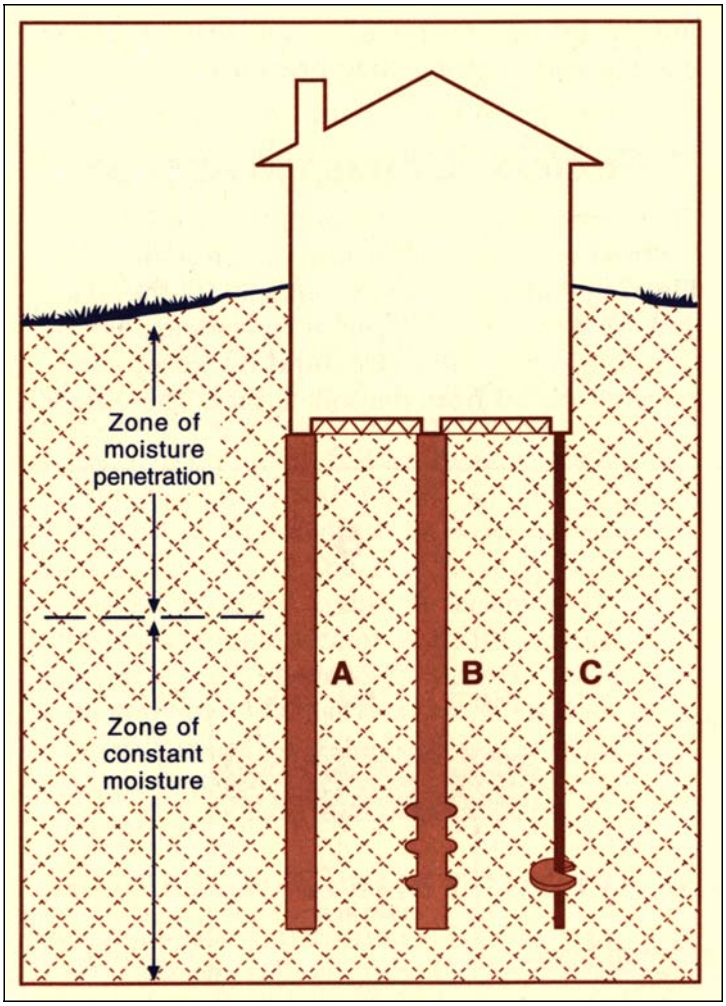

Figure 16. Three types of drilled piers commonly used in Colorado. (A) Straight-shafted concrete pier. (B) Concrete pier with grooves near base. (C) Helical steel pier. All piers should extend well below the anticipated zone of moisture penetration. From Noe and others (1997).

Several types of drilled piers are used in Colorado (Figure 16). They may be straight-shafted or may have grooves cut near the base of each pier. End-bearing drilled piers are drilled into bedrock, at least for the lowermost several feet. The load-carrying capacity of the pier is developed against a socket of "stable" bedrock at the bottom of the pier (Not all bedrock is stable, though, especially in areas having steeply dipping bedrock). Friction piers are drilled in thick soil deposits where the underlying bedrock is too deep to be reached. The load-carrying capacity of the pier is developed by friction along the shaft of the drilled pier. Helical steel piers are most commonly used in Colorado as a remedial installation to replace previously damaged foundation elements. Helical piers consist of a steel shaft with auger-like blades near the tip, which is advanced into the ground by rotation until it meets a prescribed torque resistance or depth.

Foundation Walls – Foundation walls may require reinforcement or additional supports to resist lateral pressures exerted by the adjacent soil and backfill. This is especially true when the soil and backfill are composed of swell-prone clays. The exact design depends on the length, height, and configuration of the walls, as well as soil and subsurface water conditions. Steel bars, beams, or wing-like walls (buttresses or counterforts) that extend outward from the foundation wall at a right angle may be used to provide reinforcement. An improperly designed wall is at risk of buckling or bowing inward when exposed to soils that have moderate to very high swelling pressures.

Floor Construction Designs

There are two primary types of floors used in Colorado when expansive soils are present. Floating slab floors lie in contact with the soil and are designed to accommodate some amount of soil heaving, while structural floors are completely isolated from the soil surface. These floor systems are used for basements in many areas, especially in the Front Range urban corridor, but may be used for at-grade construction in cases where basements are not used.

Floating Slab Floors – This is the oldest type of flooring designed specifically for expansive soil. These floors usually consist of a non-reinforced, concrete slab that rests directly on soil or fill (Figure 17). The slab is isolated from the outer foundation walls by a slip joint. This design allows the floor to undergo 2 to 4 in (5 to 10 cm) of vertical heaving without causing appreciable damage to the rest of the building. Special interior construction techniques, discussed later in this paper, are necessary when floating slab floors are used.

Floating slab floors perform well for soils that are non-swelling or have low to moderate swell potential. They are also commonly used in conjunction with overexcavations, where a thick layer of non- to moderately swelling material separates the slab from the underlying soils. However, floating slabs installed directly upon highly expansive soils may undergo significant heaving, cracking, and buckling. This is because they do not weigh enough to resist the uplift pressure generated when the soils are wetted. Floating slab floors are especially prone to cracking and buckling caused by uneven ground movements in areas of steeply dipping bedrock.

Structural Floors – This design, consisting of wood or composite decking supported on wood or steel-beamed floor joists, has been used increasingly in Colorado since the mid-1980s. The floor assembly is supported by the outer foundation walls, and is suspended above the soil (Figure 17). The weight of the floor and all objects on the floor is transferred directly to the foundation, thus increasing the foundation's resistance to heaving. A shallow crawl space, at least 18 in (46 cm) high, is created between the floor and the soil surface, which allows owner access and inspection. Proper ventilation is needed to reduce humidity and wood rot, mold, mildew, and deterioration. Passive or active ventilation systems should be built into buildings with structural floors, in accordance with the governing building code. Structural floors are most often used in areas where soils have high to very high swell potentials. The higher initial cost of a structural floor may be offset by better long-term performance (as compared to floating slab floors) in those areas.

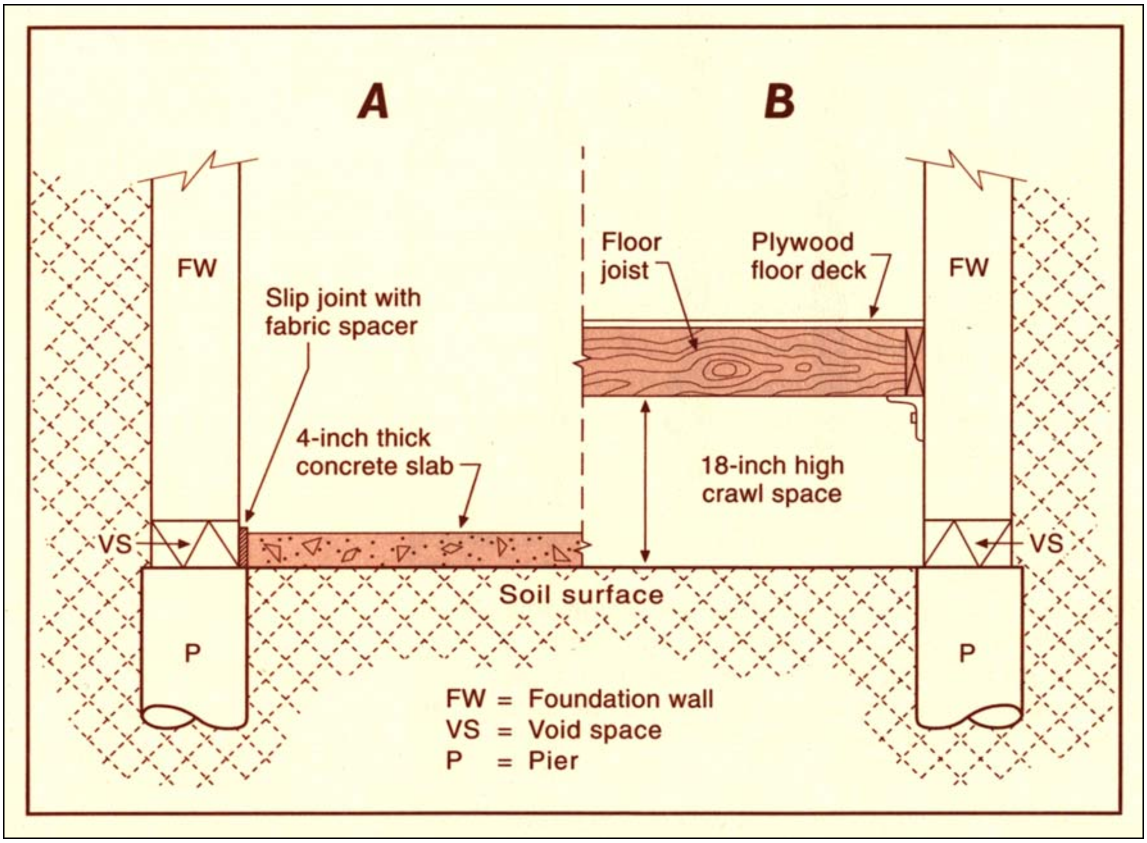

Figure 17. Two types of basement floor systems. (A) Floating slab floor. (B) Structural floor and crawl space. From Noe and others (1997).

In some cases in Colorado, builders have installed floating slab floors even though the geotechnical engineer may have originally recommended a structural floor. Many engineering reports allow for this option at the discretion of the owner (who, at that time, may be the developer or builder). An owner may choose this higher-risk option because a floating slab costs several thousand dollars less for materials and installation, a savings that may be passed on to the homebuyer. However, the homebuyer may eventually incur the cost of repairs for damaged slab floors, and any damage to the rest of the house, resulting from slab heave that occurs after the builder's warranty expires.

Interior Construction Designs

Special interior construction is necessary for any house built on expansive soil. The actual designs may vary depending on the type of foundation and flooring in the house, as well as the degree of swell potential of the soil. The basic considerations are the same regardless of whether or not the house has a basement. Many of these designs were developed for use with floating slab floors, where it is assumed that the floor will heave or settle independently of the rest of the house.

Interior Walls – Non-load-bearing interior walls used with floating slab floors commonly employ a gap or void constructed at the bottom of the wall so that it is suspended a specified distance above the floor slab (Figure 18). Should the floor heave, the floor and interior wall will shift toward each other and reduce the void, but no damage should occur as long as some void remains. However, in cases where the amount of heave has exceeded the partition void space, placing the wall in direct contact with the floor slab, deformation and damage may be transmitted to the interior wall and other parts of a building. Load-bearing interior walls may be affected by heaving of the foundation, and they may transmit deformation and damage to other parts of the building.

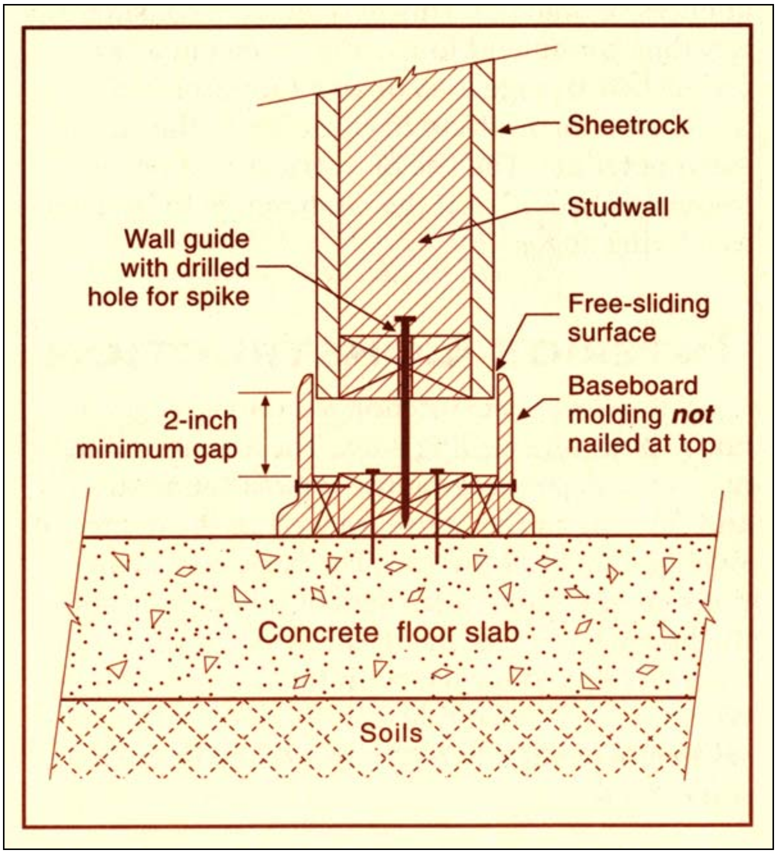

Figure 18. Detail of the bottom part of suspended, non-load-bearing interior wall. From Noe and others (1997); modified from Holtz and Hart (1978).

Doors, Windows, and Stairs – Doors and windows may be significantly affected by expansive soil. Their frames may be deformed to a point where they bind and do not open easily, or they may be rendered totally inoperable. Ideally, door and window frames should be designed with some amount of void or headspace to allow for adjustment in the event of heaving. Stairs supported on floating slab floors should not have fixed connections. An accepted design is to attach the top of the stairway to the house frame by means of a strap connection. The base of the stairway rests on the slab floor but is not connected to it. This design allows the stairway to rotate up or down to accommodate a certain amount of floor movement.

Gas, Water, and Sewer Lines – Natural gas, propane, water, and sewer lines should be designed so that they are completely isolated from floating slab floors, structural floors, and foundations. Ruptured pipes may result if the pipes are rigidly attached to a floor or foundation that heaves. The hazard resulting from the rupture of a natural gas or propane pipe is serious in terms of human health and safety. Ruptured water and sewer lines, while they do not directly affect human safety, may have a significant effect on the stability of a building. This is because water leaking from the ruptured pipe may infiltrate the ground and cause additional soil swelling and heaving.

Furnaces – Furnaces mounted on floating slab floors may be crushed between the floor and ceiling framing in the event of heaving, unless special precautions are taken. A properly designed furnace in this case will have a flexible and collapsible cowling, or boot, in the ductwork at the top. If significant heave occurs, the boot will shorten but the furnace system will remain operable.

Exterior Flatwork Designs

Most exterior flatwork (i.e., driveways, sidewalks, patios, and porches) is constructed with unreinforced concrete. Flatwork on moderately to very highly expansive soils should be designed and constructed with adequate strength according to the site's soil characteristics. In some cases, the concrete must be formulated to resist corrosion and deterioration due to alkaline chemistry of the ground water and soil. Unfortunately, concrete slabs cannot be designed to resist vertical heaving because uplift pressures exerted by expansive soils can greatly exceed the weight of the slab. In many areas of Colorado, exterior flatwork is likely to undergo some heaving and cracking in areas of expansive soils.

Concrete porches and patios may require their own drilled pier foundations, or some other form of support, to avoid heaving, tipping, or settling. Porches supported directly on expansive soils may react seasonally, rising as the soils become wet during late winter and spring and sinking as the soils dry out later in the year. They are also susceptible to settling due to consolidation and settlement of the underlying backfill adjacent to the house foundation, and to frost heaving.

Asphalt can be used as an alternative to concrete flatwork, especially for driveways. The asphalt is generally more flexible than concrete. However, asphalt driveways and walkways may still be prone to cracking due to swelling of soils with moderate to very high swell potentials, and they may require a great deal of maintenance.

Subsurface Drainage Systems

One of the most important means of reducing the risk of expansive soil damage is to control the amount of moisture that infiltrates the soil. Structures built on expansive soils should have adequate subsurface drainage systems installed, to remove excess free water that moves through the soil. Such systems can be effective in reducing expansive soil damage, although they will not eliminate the overall increase in soil moisture that occurs after development.

Perimeter Drain – Subsurface drainage around the foundation is achieved by installing a perimeter drain near the base of the foundation. This system consists of a trench (either inside or outside of the foundation wall) that contains a drain pipe; coarse, clean gravel; a geotextile drainage fabric or perforated roofing felt as a particle filter; and backfill material (Figure 19). The highest level of the drainpipe should be several inches below the level of the floor slab or base of

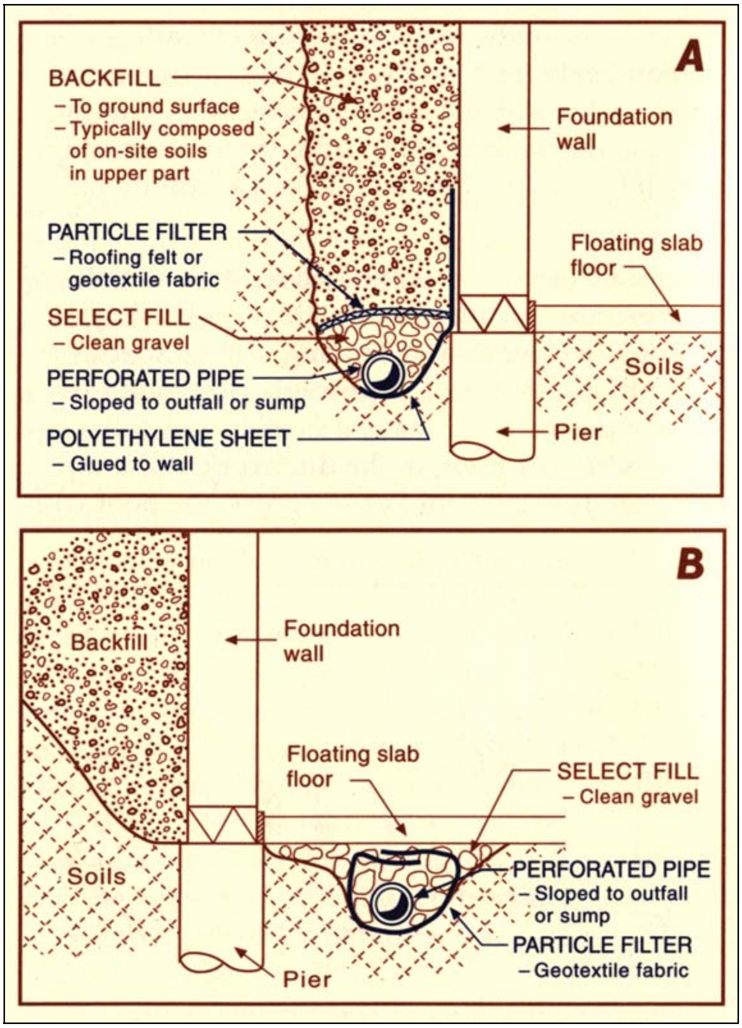

Figure 19. Components of a typical perimeter drain. (A) Exterior. (B) Interior. From Noe and others (1997).

the foundation wall. Perimeter drains should be installed with a slope of 1/8- to 1/4-inch per foot (1 to 2 cm per m) so that gravity will control the flow of the water. The drain must discharge into a sump, an area underdrain, or a suitable gravity outlet. The down-gradient extension of a perimeter drain should not terminate beneath the yard and discharge directly into the soil.

Drainpipes are made of perforated metal or plastic. Plastic pipe is generally preferred because it resists corrosion, and can be either flexible or rigid. The pipe may be slotted on all sides, or it may have two rows of opposing perforations that should be placed facing the sides of the trench. Pipes with large perforations should be wrapped with a fabric membrane to reduce clogging.

Sump System – A sump is an enclosed pit or low area that collects water. Water from perimeter-drain systems flows to the sump by gravity drainage. Water that collects in the sump is removed by an automatic submersible pump and discharged into an acceptable area. Sump pits having non-perforated bases are best for areas of expansive soil, because they keep the water from entering the surrounding soil. Sumps are usually installed in a basement. As an alternative, they can be installed outside in the yard. The effectiveness of the sump system may be reduced if larger amounts of water constantly flow into the sump, as may be the case in sandy soils or fractured bedrock having shallow ground-water conditions.

Interceptor Drain – Interceptor drains are used to collect subsurface water and divert it to an acceptable outfall. This type of drain is often used when the source of water is uphill from the area to be protected. Historically, they have been used in Colorado to protect individual houses or small neighborhoods from seepage from unlined irrigation canals. A typical interceptor drain consists of a gravel or sand-filled trench, either with or without a drainpipe. It may be lined with a permeable fabric membrane to help prevent clogging, or it may include an impervious membrane on the downhill side of the trench.

An area drain is similar in construction to an interceptor drain. Typically, these drains run beneath streets. They gather subsurface water from perimeter drains of individual houses and other sources (e.g., excess irrigation or leaking water and sewer lines) and divert it to a gravity outfall (Figure 20). The trenches for the drain are typically dug down to below the level of other utility lines, and the upper part of the trench is most often filled with compacted native backfill. Area drain systems are common in newer subdivisions along Colorado's Front Range as an alternative to individual sump systems. They have the advantage of intercepting numerous sources of subsurface water from a relatively large area. The system must be maintained and inspected regularly, because covering or clogging of the outlet may lead to widespread water build-up and possible expansive soil damage.

Septic Systems – Septic systems with leach fields are often installed for houses in rural settings. Leach fields are a source of liquids that infiltrate the ground, and therefore should be located well away and down slope from buildings if expansive soil is present. Proper siting and design of leach fields is necessary so that any resulting perched water does not flow toward or cause wetting of the soil around any nearby houses.

Surface Drainage Systems

Proper surface drainage is critical for structures built on expansive soils. Water from rainfall, snowmelt, and irrigation must not be allowed to pond and infiltrate the soil near foundations or flatwork. Instead, it must be directed into drainage swales and carried away from the property by means of ditches, street gutters, storm drains, or other available means.

Roof Drainage – The roof drainage system is composed of gutters, downspouts, and splashblocks (Figure 21). Its purpose is to keep rainwater and snowmelt from pouring or dripping over the eaves and falling next to the foundation. Fixed downspout extensions and splash blocks are two means of carrying water away from the house beyond the backfill area. All roof runoff should be carried at least 5 ft (1.5 m), and preferably 10 ft (3 m) away from the building.

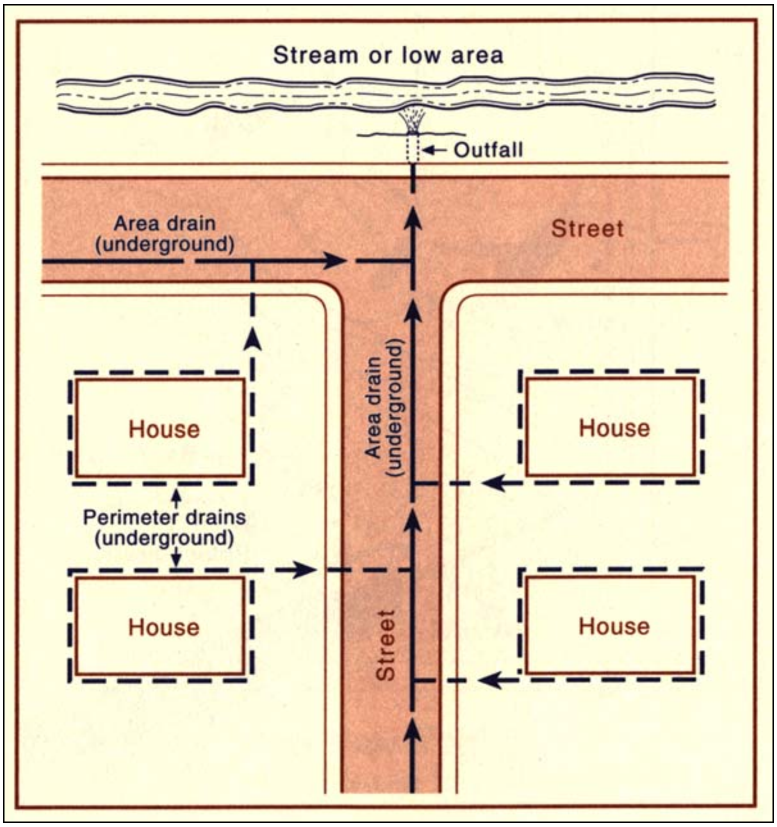

Figure 20. Map view of a typical area drain layout. From Noe and others (1997).

Slope Drainage – A properly designed and maintained slope next to a structure is a critical aspect of surface drainage. The slope and adjacent ditches and swales should be graded according to the specifications of a qualified engineer. The main purpose of lot grading is to provide positive drainage away from the structure. If the lot is sloping and well drained, precipitation will run off and infiltration near the foundation will be reduced.

The minimum slope or fall necessary within 10 ft (3 m) of a building depends upon the type of surface and/or landscaping. In paved areas, a minimum slope of 1 percent (1-2 inches vertical fall for 10 feet of horizontal distance) should be maintained. A greater, as-built initial slope of 2-5 percent may be desirable, however, as even a small amount of backfill settling can reverse a small slope and cause water to pond. For landscaped runoff slopes (Figure 22), the fall of the slope should be at least 10 percent (1 ft vertical to 10 ft horizontal) (10 cm vertical to 1 m horizontal). Many newer houses being built in Colorado, especially on small, closely spaced lots, have slopes as steep as 33 percent. Soil immediately beneath these paved and landscaped slope surfaces should be well compacted and fine-grained so that water will not easily infiltrate the backfill. All slopes should be properly landscaped with rocks or other mulches to prevent erosion.

Ditches and Swales – Runoff water from roof and slope drainage systems can be collected and carried away from the house by ditches and swales. These shallow trenches or depressions in the yard are graded to collect, direct, and convey rainwater, snowmelt, and excess irrigation water

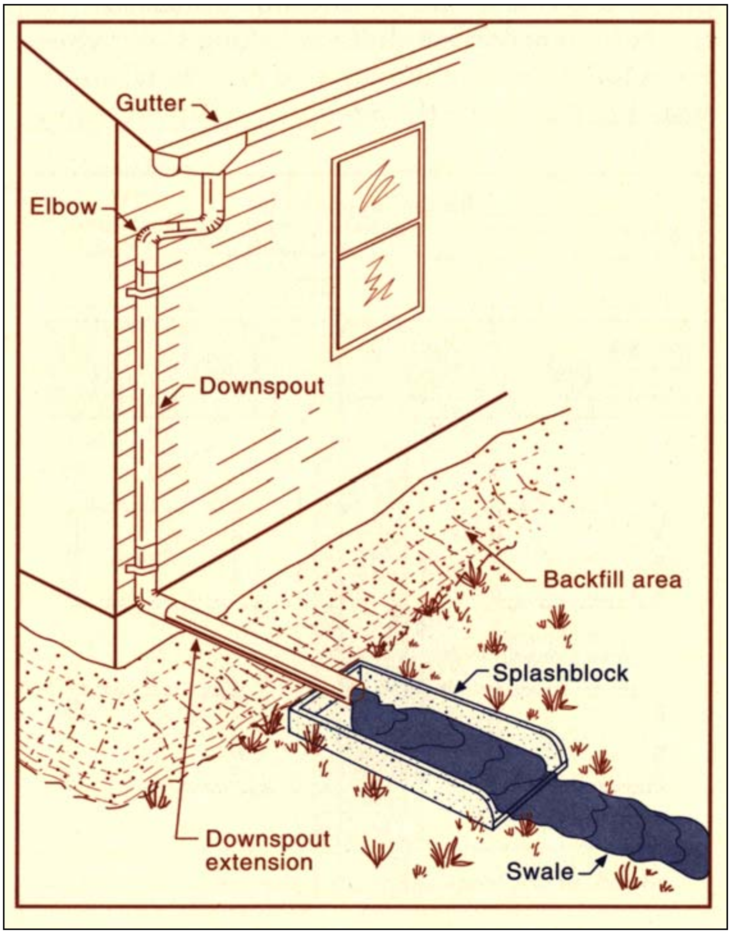

Figure 21. Components of a roof drainage system. From Noe and others (1997); modified from Jochim (1987).

away from the house and off the property. Care must be taken to ensure that the surface water channeled away from a structure is not directed toward neighboring structures. Ditches and swales may drain into commonly shared concrete gutters and storm sewers in suburban areas. In rural areas, culvert pipes are installed along ditches so that runoff water can flow under roadways.

Construction Quality Control

Quality control is perhaps the most important aspect of construction, especially in areas of expansive soil. Even though soil and water conditions may be initially responsible for expansive soil movement, poor construction quality can add significantly to the total amount of damage to a house. Any one of the construction designs and methodologies described previously may be rendered useless unless it is done carefully and correctly.

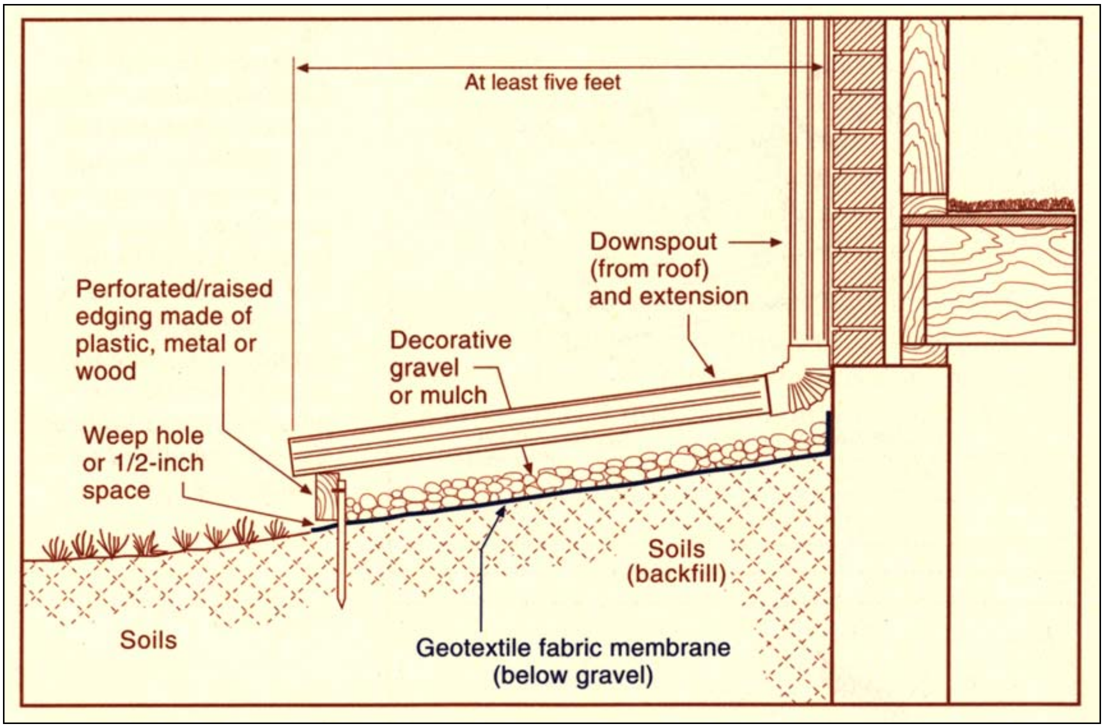

Figure 22. Landscaped runoff slope next to a house foundation. The roof drainage is carried by a downspout extension to a point beyond the slope. From Noe and others (1997); modified from Holtz and Hart (1978).

LANDSCAPING AND MAINTENANCE PRACTICES

Guidelines for Landscaping

The landscaping conventionally used in Colorado consists of luxuriant bluegrass lawns, showy gardens, and large shade trees. Many of these plants originated in more temperate climates, and have water requirements that cannot be satisfied by rainfall alone. Residents augment the natural precipitation with large amounts of water using irrigation. As a result, the soil beneath a property usually takes on additional or excess water. Most geologists and engineers who work with soil in Colorado agree that excess water is the most significant and direct cause of expansive soil damage.

Landscaping on expansive soil should be geared toward reducing the amount of excess water that infiltrates the ground, especially in the immediate area around the house foundation. Some basic guidelines are:

1) Do not plant flowers or shrubs closer than 5 feet from the foundation, unless they have very low water requirements and are hand- or drip-line watered.

2) Plantings near the foundation should not disturb the runoff slope around the building. Roof runoff should be directed away from the slope and foundation, not into the plantings.

3) Trees should not be planted closer than 15 feet from the foundation. Trees with high water requirements or extensive, shallow root systems (e.g., willows, poplars) should be avoided.

4) Sprinkler systems should not spray water any closer than 5 feet from the foundation. Irrigation should be limited to the amount necessary to keep plants healthy.

5) Group plants according to similar water needs so that different areas of vegetation can be irrigated in a water-wise manner.

6) Use low-water vegetation throughout your property, including gardens and lawns. (See the discussion of Xeriscape™ landscaping, below.)

7) Water existing trees near to a house during long, dry periods. This will keep them from extending their root systems and drawing large amounts of water from the surrounding area.

8) Poor-quality, "heavy" clay soils should be improved and conditioned by mixing in organic material to improve the fertility, aeration, and water circulation within the topsoil.

Landscaping with Xeriscape™

Landscaping conditions in Colorado are different from most other parts of the country. The state's high elevation and semi-arid climate give rise to a short growing season, low precipitation (at least around the major population centers), and occasional droughts. The soils tend to be alkaline, and are calcium-rich on the Eastern Slope and sodium-rich on the Western Slope. The clay soils are characterized by having poor aeration (air circulation) and poor drainage. Another serious constraint is the large amount of water needed to grow a conventional lawn and garden.

Xeriscape™ is a practical solution to landscaping under these seemingly unfavorable conditions. Pronounced "Zeer'-is-scape," the term means, "water-wise landscaping" (from "xeros", the Greek word for dry), and was coined by the Denver Water Department (now Denver Water). Xeriscaping is a process aimed at conserving water, based on proper planning and design, use of mulches and/or turf alternatives, zoning of plants, soil improvements, efficient irrigation, and appropriate maintenance (Xeriscape Colorado, Inc., undated pamphlet).

A Xeriscape requires little maintenance after it is established. There is a dramatic difference in the water demands of a conventional bluegrass lawn versus Xeriscape plantings (Figure 23). Colorado homeowners have been able to reduce their total household water use by as much as 50 percent by installing water-wise landscaping (Denver Water Department, 1988). A wide variety of plant types may be used (Denver Water, 1996). An important, indirect benefit of Xeriscaping is that it can help to reduce expansive soil damage to a home

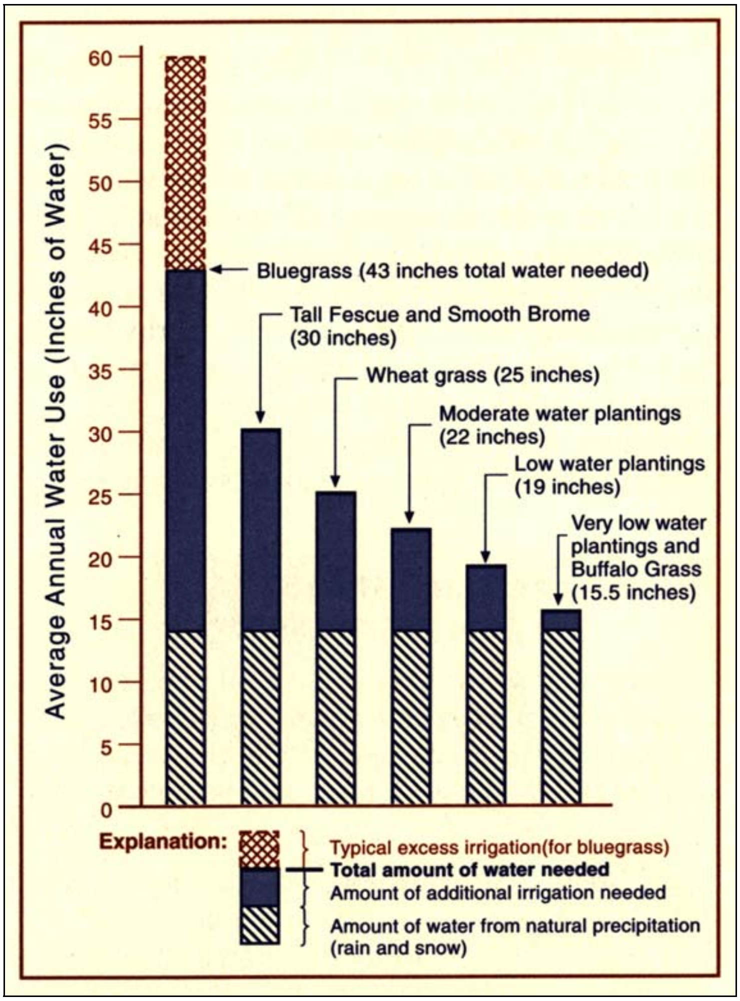

Figure 23. Average annual water use for different types of plants for an area having 14 inches of natural precipitation. The native grasses are a water-wise alternative to conventional bluegrass lawns. From Noe and others (1997); modified from Xeriscape Colorado!, Inc. (undated pamphlet).

Mulch ground covers are an important component of Xeriscaping, especially for areas of expansive soil. A mulch landscape consists of two parts, a geotextile fabric base and an overlying mulch cover. A good-quality geotextile fabric will control weeds and retard infiltration, but will still permit evaporation. The use of impermeable plastic sheeting is discouraged because it prevents normal evaporation from occurring. Mulches can be organic (bark, wood chips, etc.) or inorganic (boulders, cobbles, gravel, or crushed rock). An attractive, relatively low-maintenance landscape can be created by using gravel edgings, rock gardens, low-water ground cover, and a limited central area of lawn (Figure 24).

Figure 24. Examples of landscaping using mulch ground cover, where the central lawn area is accented by gravel edging. From Noe and others (1997).

Maintenance Practices

The lack of timely maintenance to slope, drainage, and landscape areas around a building or other facilities can contribute significantly to expansive soil damage. Severe problems may result from poor maintenance practices, examples of which include the following:

1) neglecting to maintain adequate slopes for good drainage
2) neglecting to clean gutters and downspouts
3) overwatering lawns and gardens
4) neglecting to adjust and maintain sprinkler systems
5) planting trees, shrubs, and flowers too close to the foundation
6) constructing patios, fences, or other obstructions that dam and pond water
7) neglecting to seal construction joints and cracks that develop in flatwork

It is essential that property owners understand how to check and maintain all of the different systems that were designed to protect a building against expansive soil damage.

Concrete Floors and Walls – Periodic inspections of concrete slabs and walls, both inside and outside of buildings, should be conducted. This is especially important during the first five years after construction, when the most severe adjustment occurs between the building and its environment. Although some cracking will occur in virtually all concrete slabs and walls, it tends to be more common and more severe in areas of expansive soils. Unsealed cracks may allow water to infiltrate into the ground, and could cause the cracking to worsen. Cracks should be sealed as soon as possible, using quality exterior acrylic caulking compounds or equivalent products. Crack widths can be regularly monitored by measuring at a designated spot along each crack.

Crawl Space Ventilation – Ventilation of the crawl space beneath a structural floor is essential, and contributes to the proper performance and durability of the floor. Building owners should be familiar with any maintenance and special requirements of structural floors and any attendant systems. In Colorado, there are numerous cases where moisture and humidity build-up has led to rotting and deteriorating wood and the growth of mold. Active and passive ventilation systems should be kept in good working order. If such systems are absent, and moisture and humidity build-up is occurring, consideration should be given for retrofit installations. The installation of ground-surface vapor barriers should also be considered.

Subsurface Drainage – Subsurface drains should require little maintenance if correctly installed. For gravity-discharge perimeter, interceptor, or area drain systems, it is extremely important to avoid covering or obstructing the drain at the point where it discharges. Homeowners associations should be aware of the location of area drains in a subdivision, and should have the system maintained regularly. There are numerous cases in Colorado where subsurface drains have been broken, installed incorrectly, or even not installed at all. In any of these cases, it will probably be necessary to dig up the drain in order to diagnose the problem and make the appropriate repairs.

Sump systems require periodic inspection and, if water has entered, cleaning of the sump pit and maintenance of the submersible sump pump. Perforated sump pits are not recommended in expansive soils because they allow standing water to infiltrate the surrounding soils. It may be advisable to upgrade such a system with a non-perforated sump pit.

Surface Drainage – Surface drainage systems are designed to reduce the amount of water that infiltrates into the ground, and they must be kept in good working condition. By taking the time to maintain and repair these systems, the owner will increase the life of a building and reduce the potential for costly repairs. Regular, annual or semi-annual inspections should be made of roof gutters, downspouts and extensions, splash blocks, and drainage swales. Sprinkler systems, both manual and automatic, should be checked and maintained often to prevent leakage into the ground from cracks in hoses and loose-fitting joints.

A critical aspect of surface drainage is maintaining a positive slope over the backfill area next to the building foundation. This material may settle enough to reverse or flatten the slope. Reverse or negative drainage will cause ponding and infiltration of water during precipitation or heavy irrigation. Regular maintenance involves maintaining the slope angle, re-compacting the surface soil, and sealing or replacing damaged concrete slabs (Figure 25).

Figure 25. Settling of the backfill material has caused this sidewalk slab to settle and crack, resulting in reversed drainage and ponding next to the foundation. From Noe and others (1997); from Jochim (1987).

Landscaping – Landscape plantings, mulch covers, and irrigation systems should be well maintained, especially on runoff slopes near foundations. It may be prudent to delay installing any landscaping adjacent to the foundation until the backfill has had a chance to settle. Xeriscapes may require greater amounts of watering and maintenance for the first few years after planting than is required thereafter. Periodic maintenance is still needed after the Xeriscape is established to keep weeds out and to ensure the performance of the plants and mulches.

PROPERTY OWNER RISK AND DISCLOSURE

Expansive soil constitutes a powerful and costly geologic hazard in Colorado. It is often difficult to evaluate the site-specific risks posed by this hazard, however, because each site is unique in terms of its geologic setting and swell potential, and the as-built construction and maintenance of building, drainage, and landscaping systems. Purchasing a house or property that is underlain by expansive soil requires a realization, and acceptance, of the inherent risks. Ideally, this involves an informed decision about the potential expansive soil hazard and the risks, along with other considerations such as location, lifestyle and affordability.

Expansive soil and bedrock are widespread in Colorado and are not easily avoided. Therefore, property buyers need to be aware of the distinction between the presence and potential severity of expansive soils. The mere presence of expansive soil beneath a property gives no definitive indication of the potential severity of the swelling hazard. Buyers should be more concerned about the soil's swell potential (Is it low, moderate, high, very high, or non-swelling?) and how the facilities on the property were designed and built with regard to those actual soil conditions. One should expect that expansive soil will swell and heave to a certain degree in response to development and irrigation.

New Homes – Disclosure of expansive soil hazards for new homes is required under Colorado Revised Statute C.R.S. 6-6.5-101 (Senate Bill 13, 1984). This statute describes the responsibility of a builder of a new home to disclose evidence of any significant soil hazards, including swelling/expansive soils, to a potential buyer. The disclosure requirements, from Part 1 of the statute, are as follows:

> *At least fourteen days prior to closing the sale of any new residence for human habitation, every developer or builder or their representatives shall provide the purchaser with a copy of a summary report of the analysis and the site recommendations. For sites in which significant potential for expansive soils is recognized, the builder or his representative shall supply each buyer with a copy of a publication detailing the problems associated with such soils, the building methods to address these problems during construction, and suggestions for care and maintenance to address such problems.*

There are no criteria in the statute for determining "significant" potential for expansive soils. In practice, the potential may be seen as "significant" when the project geotechnical engineer recommends using certain construction methods and designs specifically to reduce the effects of expansive soils. This information should be included in a summary soil report for each lot or for a larger project. Ideally, a summary soil report should include the swell potential, observations, and recommendations given for the subject home-site. The information provided should be the most specific information available for the site. It should include the engineering information used by the builder or developer in determining the site's building recommendations.

Resale Homes – Buyers of resale homes in Colorado are also protected by disclosure legislation. Real estate brokers are required to disclose all adverse material facts under the provisions of C.R.S. 12-61-801 et seq. (Senate Bill 223 1993). The presence of expansive soil, although not specifically named, may be considered an adverse material fact, because it can affect the physical condition of or cause defects in the property. A violation of disclosure requirements by the real estate broker may be investigated by the Colorado Real Estate Commission under C.R.S. 12-61-113(1).

Sellers of a resale home should be asked to fill out form LC18-9-95, Seller's Property Disclosure, which specifically lists the presence of expansive soil as a hazardous condition (Part 4). This form is supplied by the real estate broker and was created by the Colorado Real Estate Commission (Colorado Division of Real Estate, 1995). Both buyer and seller sign the form as part of a property sale. Non-disclosure of adverse material facts by the seller may constitute misrepresentation or fraud, and is covered by common law.

It may be possible to determine if expansive soil has affected a resale home by looking for telltale signs of damage and/or repairs. In answering hundreds of inquiries about home inspection by the general public, the CGS recommends that a qualified engineering consultant should be hired to assess the physical condition of the home, the soil report, and the foundation design. Listings of consultants are provided in the local yellow pages under "Engineers-Foundation," "Engineers-Geotechnical-Soils," or "Engineers-Structural." In addition, Hoffman (1972) and Noe and others (1997) describe step-by-step guidelines on how to check a property for expansive soil and other damage.

CONCLUSIONS

1) Expansive soil and bedrock constitute Colorado's most costly geologic hazard in terms of damage to private and public facilities. Expansive soil has caused varied levels of damage to pavements, driveways and sidewalks; building walls, floors, and foundations; and water and sewer lines in many parts of the state.

2) Smectite (montmorillonite) and mixed illite-smectite are the clay minerals associated with expansive soil and bedrock. In particular, smectite has the ability to attract water into both its interlayer and intercrystalline gaps. Calcium (Ca^{2+}) is the most common interlayer cation in the Pierre Shale, although Sodium (Na^{+}) is also present.

3) Expansive clay-bearing materials are widespread across Colorado, and are particularly common in the Jurassic, Cretaceous, and Tertiary formations and their derived Quaternary soil deposits. A majority of the state's major population centers are located in areas of potentially expansive soil and bedrock.

4) On a localized scale, the extent and the severity of the hazard depends upon a number of factors including the composition, engineering properties, and three-dimensional framework of geologic units underlying a site. The natural and eventual ground-moisture profile of a site is an especially important consideration.

5) The risks associated with expansive soils and bedrock can be reduced, but not eliminated, by careful design and construction procedures. However, one should expect that expansive soil will expand and heave to some degree in response to development and irrigation.

6) The potential severity of damage due to expansive soils can be significantly reduced if steps are taken to recognize the problem and then design, construct, landscape, and maintain the home in a responsible manner (Figure 26A). However, leaving out or cutting corners on any one of these steps can lead to dramatic and devastating results (Figure 26B).

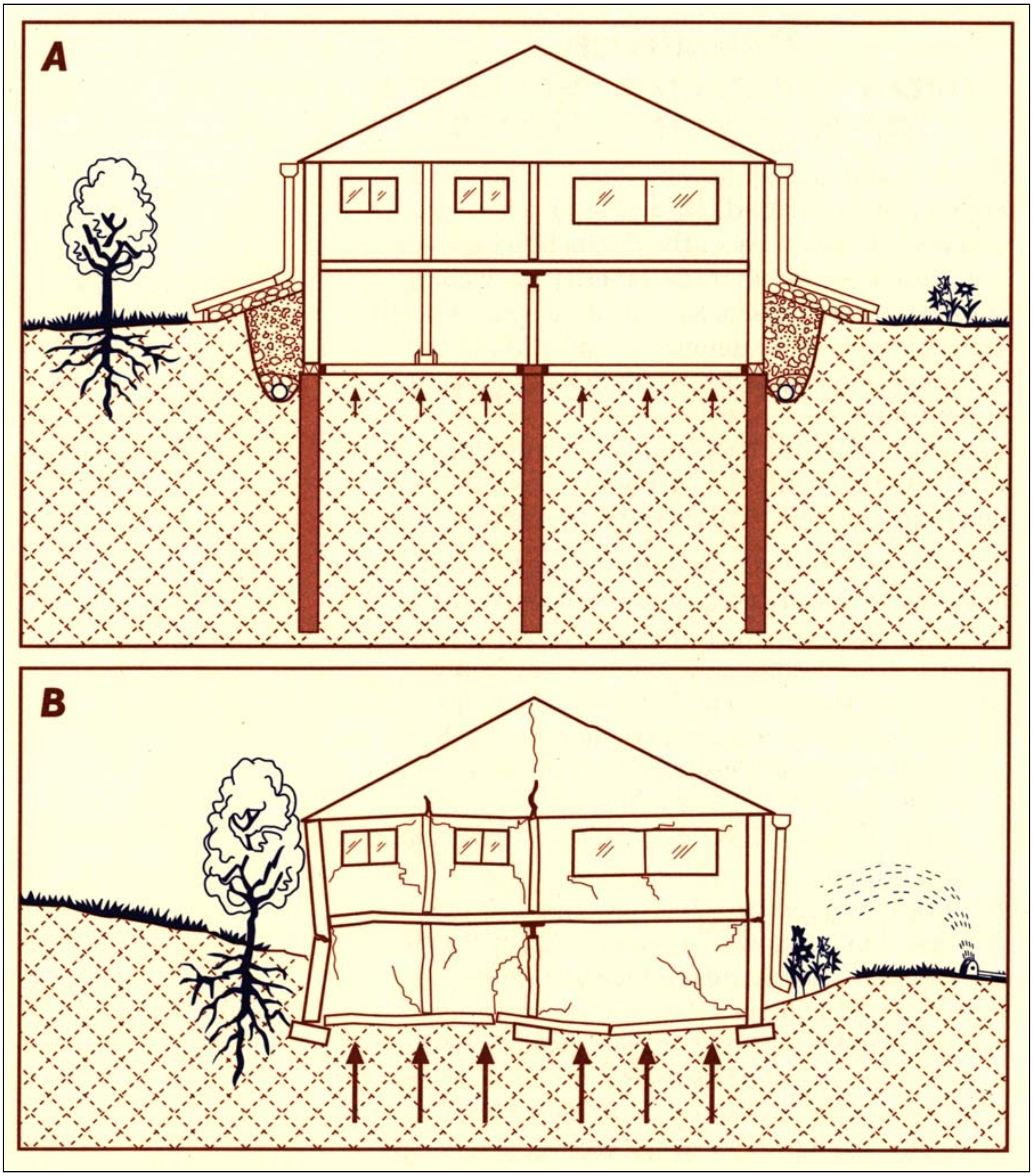

Figure 26. The results of (A) proper versus (B) improper design, construction, landscaping, and homeowner maintenance for homes built on expansive soil. From Noe and others (1997); modified from Holtz and Hart, (1978).

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TRENCHING INVESTIGATIONS IN EXPANSIVE, STEEPLY DIPPING BEDROCK IN COLORADO

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Key Terms: expansive soil, swelling soil, expansive bedrock, bentonite, heaving bedrock, differential heaving, geologic hazards, site exploration, trenching

ABSTRACT

Site investigations in areas underlain by steeply dipping bedrock require trenching, in addition to boreholes, as means of characterizing local bedrock heterogeneity and assessing the potential for differential heaving. Trenching has become a mandatory part of the development planning process in some counties along Colorado's Front Range foothills. The goals of an integrated borehole/ trenching investigation are to (1) use conventional geotechnical borings to assess the depth of bedrock at different locations and to sample and test the overlying soil materials; (2) identify potential trenching sites by determining areas where the bedrock lies at shallow depths beneath the ground surface; (3) expose a continuous section of near-surface, dipping bedrock in a trench dug perpendicular to bedding strike; (4) log all pertinent beds, zones, and discontinuities; (5) recover samples from critical areas along the trench wall for laboratory testing; and (6) analyze the test results in a way that emphasizes spatial relevance. Differential heaving is likely where there are adjacent beds or zones of bedrock having different swell characteristics, and along bedding planes and shear zones where inter-block movements have occurred in the geologic past. The results of the site investigation are used to help determine whether intensive mitigation methods such as deep sub-excavation and fill replacement are needed.

INTRODUCTION

Expansive, steeply dipping bedrock (also known as heaving bedrock) constitutes a distinct geological hazard where the sedimentary bedrock layers are upturned and tilted. In such areas, the bedrock layers or blocks may swell unevenly to form parallel, linear heave features along the ground surface (Figure 1). Public and private facilities built over such heave features may be subjected to extreme amounts of vertical and lateral stress, and the resulting damage can be severe. This hazard is responsible for tens of millions of dollars in excess maintenance costs to homeowners, utility companies, counties or municipalities, and taxpayers since the 1970s when large-scale development began in certain areas of Colorado (Noe, 1997). This geological hazard also occurs in other parts of the United States, particularly near the populated areas of west-central and southern California (Meehan and Karp, 1994; Meehan, 1999).

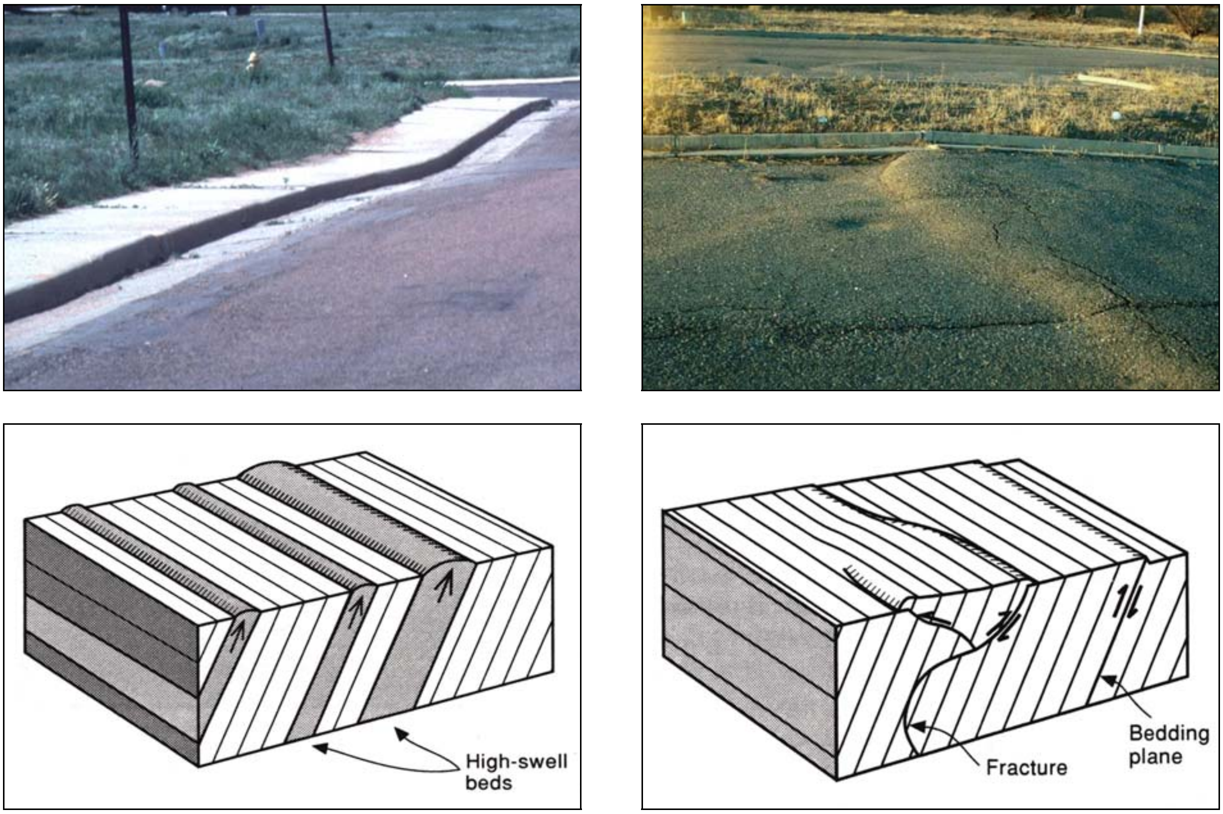

Figure 1. Examples and models of differential ground heaving caused by swelling of individual bedrock layers (left) and shear-slip movements along bedding planes or fractures (right). Block diagram models from Noe and Dodson (1999).

In Colorado, steeply dipping bedrock is found in the sedimentary formations that flank many of the mountain uplifts across the state. Structurally, these areas comprise the steep limbs of monoclinal folds; the same formations become less steep to flat lying away from the fold axis (Figure 2). Heaving bedrock hazards occur where these steeply dipping formations contain expansive clay minerals such as smectite, at least in part. The mechanisms responsible for heaving bedrock movements are geologically complex. Heaving may occur due to swelling of individual bedrock layers, each having a different swell potential, or due to shear-slip movements along bedding planes or fracture surfaces (Figure 1). Individual heave features may attain sizes as large as 2.4 ft (0.7 m) high, several tens of feet (several meters) wide, and several hundreds to thousands of feet (tens to hundreds of meters) long.

Many construction designs commonly used to mitigate the impacts of flat-lying expansive soil, such as drilled pier foundations, have met with limited success in areas of differentially heaving bedrock. One method that may counteract the differential heaving is overexcavation and fill replacement (also called deep sub-excavation), whereby a house is isolated from the bedrock by a thick pad of engineered fill. Jefferson and Douglas counties, located along the Front Range foothills near Denver, now require more detailed site investigation and specialized building techniques where heaving bedrock conditions exist. These areas are defined in overlay maps that show the extent of heave-prone, steeply dipping bedrock formations. For more information about

Colorado's dipping bedrock problem and its solutions, see Noe (1997), Noe and Dodson (1999), and papers in this volume by Noe (2003) and McOmber and Glater (2003).

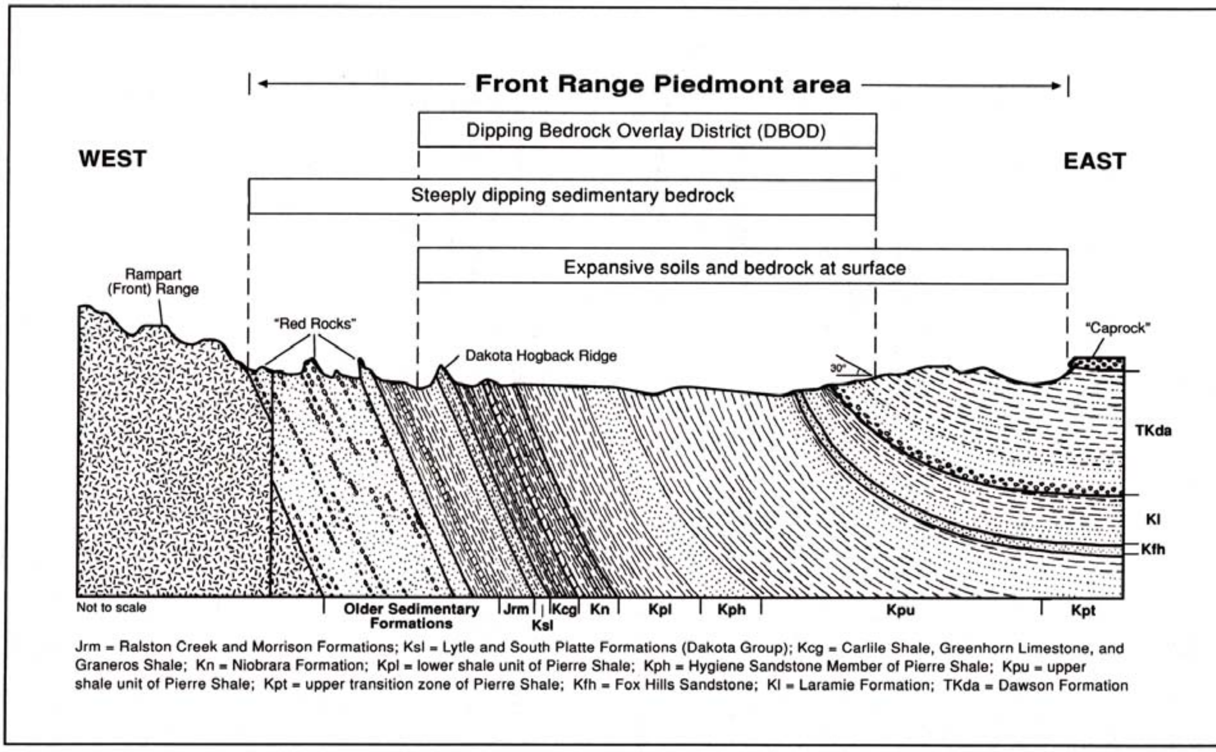

Figure 2. Schematic cross-section of geologic formations in northwestern Douglas County, showing expansive and steeply dipping bedrock zones. From Noe and Dodson (1999).

The purpose of this paper is to describe geotechnical site investigations and the use of trenching to characterize heaving bedrock hazards in areas of steeply dipping bedrock. This includes a discussion of minimum standards required by Jefferson County, which have become the standard of geotechnical practice, and practical guidance for trench logging. Many of these discussions are modified from Colorado Geological Survey (CGS) Special Publication 42, "Heaving Bedrock Hazards Associated with Expansive, Steeply Dipping Bedrock, Douglas County, Colorado" (Noe and Dodson, 1999). This booklet is also the source for many of the figures used herein.

CONTROLS OF DAMAGE

The actual distribution and magnitude of heaving damage from expansive, steeply dipping bedrock is often variable on a local scale. A particular subdivision may contain areas that are significantly affected and other areas that are apparently unaffected. The damage appears to be controlled by a number of interrelated, geological and non-geological factors.

Geological Factors

Studies of heaving bedrock by the Colorado Geological Survey and others show that damage will most likely occur where the near-surface bedrock is steeply dipping (greater than 30 degree inclination), composed of expansive claystone (at least in certain layers), and initially "dry" in its natural state. In general, the occurrence of heaving bedrock is a function of bedrock structure (bedding dip, folding, faulting, fracturing), sedimentology (formation stratigraphy, composition, and bedding continuity), loading and unloading history (degree of overconsolidation, overburden thickness), and moisture characteristics (bedrock moisture content, depth to water table).

The overburden thickness is a particularly important control. Thompson (1992a, 1992b) found that 10 ft (3 m) or more of overburden, consisting of either natural soil or engineered fill, beneath the base of a foundation is required to achieve satisfactory foundation performance in areas of heaving bedrock.

Heaving bedrock damage is most pronounced within 1 to 3 mi (1.6 to 4.8 km) of the mountain front, in heavily populated suburban areas along the Front Range foothills southwest of Denver in Jefferson and Douglas Counties, and in the western part of Colorado Springs in El Paso County (Figure 3). The Cretaceous Pierre Shale is the most prevalent, and heave-prone, sedimentary bedrock formation in this area. However, there is evidence that other formations are capable of undergoing differential heave where they are steeply dipping. These include the Pennsylvanian Glen Eyrie Shale Member of the Fountain Formation, the Jurassic Morrison Formation, and the Cretaceous Graneros Shale, Greenhorn Limestone, Carlile Shale, Niobrara Formation, Laramie Formation, Arapahoe Formation, and Denver Formation.

Many of these formations contain discrete layers of bentonite, altered volcanic ash that has been transformed into pure or nearly pure smectite claystone. Bentonite beds are often very highly expansive. In east-central Colorado, such beds are typically less than 12 in (30.5 cm) thick and make up only a minor part of the formations' overall composition, but they exert an enormous influence on swelling behavior. More typically, most of the fine-grained shales consist of silty claystone to clayey siltstone, containing mixed illite-smectite clays of varying proportions. The shales may be highly expansive as well.

On Colorado's western slope, heaving bedrock hazards may occur where formations such as the Cretaceous Mancos Shale, Mesaverde Group, and Lewis Shale are steeply dipping. Fewer problems of this type occur in western Colorado. This is probably attributable, in part, to a lack of extensive suburban development over these outcrop areas. Also, these formations are typically more sandy and silty and less clayey, as they were deposited closer to the Cretaceous shoreline, and are therefore less highly expansive than the time-equivalent formations to the east.

Non-Geological Factors

For individual houses, non-geological factors such as foundation design, construction quality control, lawn irrigation, and homeowner maintenance practices may contribute to the amount of damage that occurs in areas that overlie expansive, steeply dipping bedrock. The cumulative age of a house is also a factor; the onset of damage typically occurs within ten years after construction, and certain areas have experienced recurring ground deformations and damage for nearly 20 years since being built.

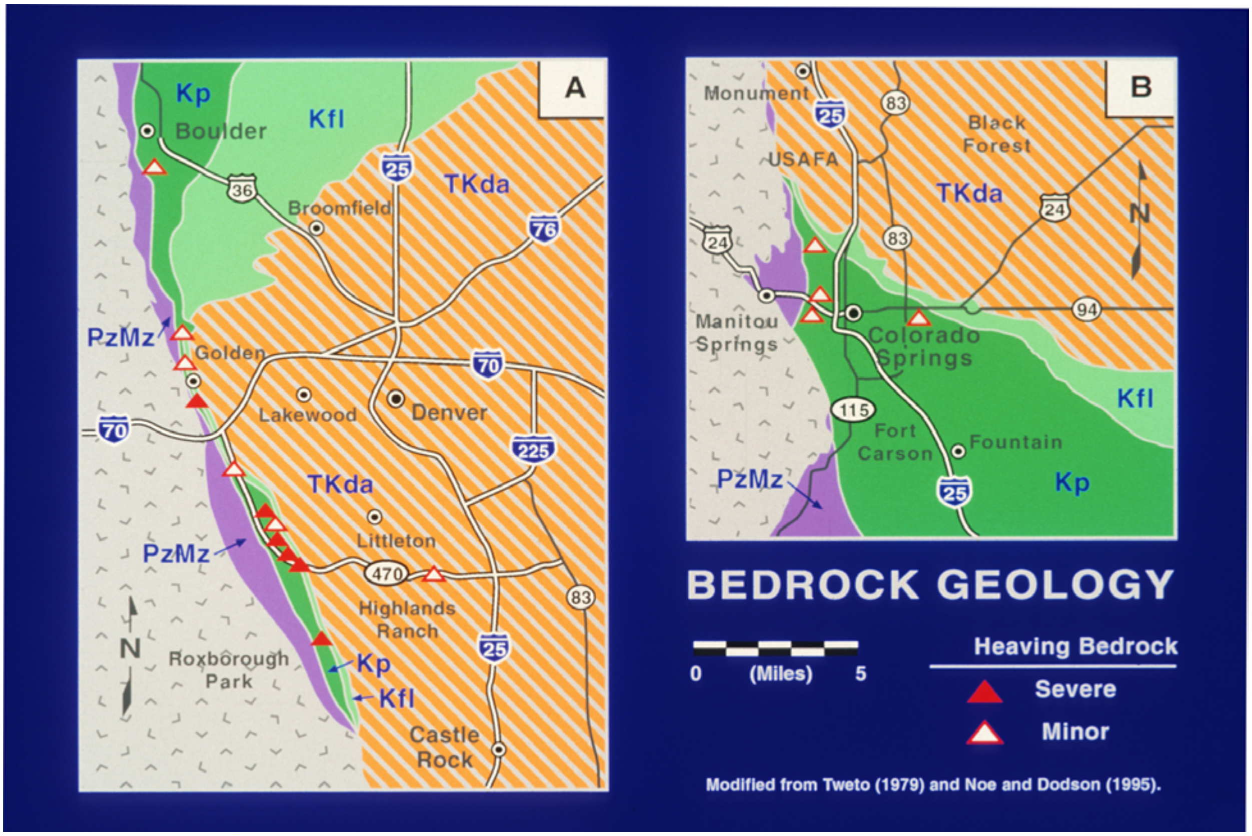

Figure 3. Simplified bedrock geology map of the Denver and Colorado Springs areas, showing general locations of damage from heaving of expansive bedrock. Mapped geologic units include Precambrian crystalline rocks (gray hatchured), undifferentiated Paleozoic and Mesozoic (PzMz), Cretaceous Pierre Shale (Kp), Cretaceous Fox Hills Sandstone and Laramie Formations (Kfl), and Cretaceous-Tertiary Denver and Dawson Formations (TKda). Modified from Tweto (1979) and Noe and Dodson (1999).

SITE INVESTIGATIONS

The purpose of a geotechnical site investigation is to determine the presence of geologic conditions that constitute constraints or hazards to development, to characterize those conditions, and to formulate effective mitigative recommendations. Flat-lying expansive soil and bedrock are treated similarly in most cases for site-investigation purposes. In contrast, the internal composition, geometry, and structure of expansive, steeply dipping bedrock allows for complex mechanisms of expansion and movement. These differences necessitate the use of distinct and hazard-specific site investigation approaches and techniques.

Site Investigation Approaches

Samples recovered during a geotechnical site investigation are assumed to be representative of the surrounding earth materials. In soil mechanics theory, a sample is assumed to be isotropic and homogeneous in its properties. These assumptions are the basis for subsequent engineering assessments and designs. In practice, however, soil and rock profiles may have significant anisotropy and heterogeneity due to differences in composition, bedding, or other discontinuities. Among the primary responsibilities of an engineering geologist are identifying different soil or rock units that underlie a site, characterizing the three-dimensional framework of those materials, and directing the sampling such that representative samples are recovered from critical locations.

In typical practice in Colorado, boreholes are most often used to assess expansive soil hazards. Flat-lying to gently dipping bedrock is not differentiated from flat-lying soil for investigation and mitigation-design purposes. However, trenching may be more effective and is used for certain situations where the bedrock is steeply dipping.

Boreholes – Drilling is effective for relatively flat-lying soil and bedrock because it allows for inspection and sampling of successively deeper layers (Figure 4a). Samples are recovered for laboratory testing from boreholes, usually at 5-ft (1.5-m) intervals, using rig-driven California samplers or Shelby tubes. The success of the investigation depends on identifying and characterizing the different layers or zones that lie in vertical succession. Once the hole is logged and the critical geological zones have been identified, samples from those zones are tested, and the results are used as a basis for the mitigative designs. The sample is regarded as being representative of the surrounding lithology at that depth, although this assumption may be complicated by a lack of lateral continuity of bedding in the soil or bedrock.

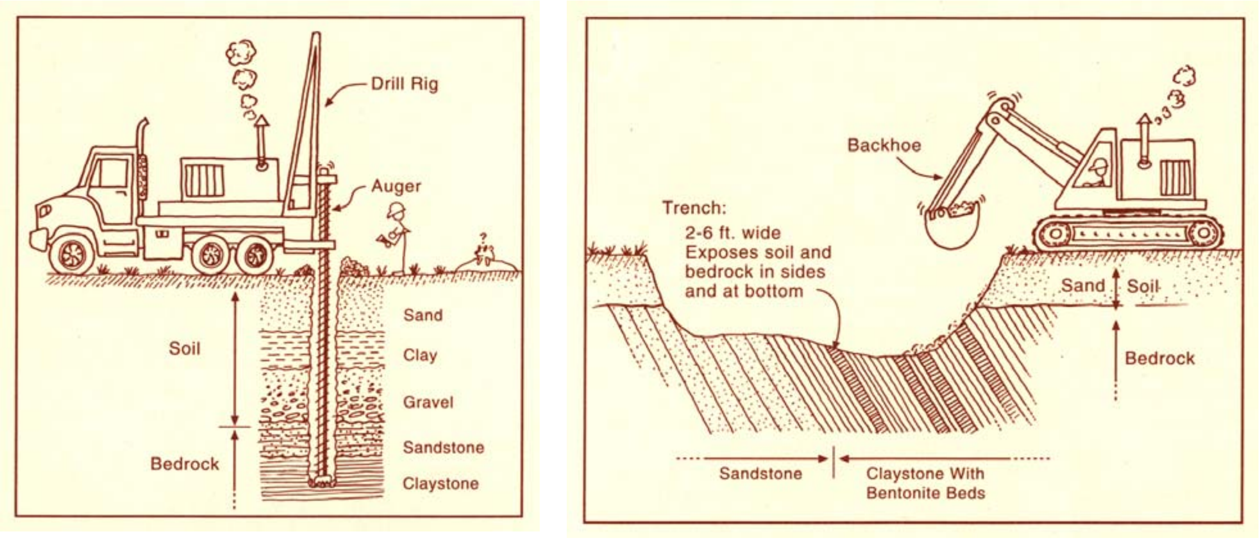

Figure 4. Methods for assessing subsurface geology: (A) Drilling. (B) Trenching. Modified from Noe et al. (1997).

Boreholes have a distinct disadvantage when the bedrock layers dip at angles of greater than 30 degrees. The geometry of the bedding is such that there is no lateral continuity of layers at a constant depth. Instead, numerous layers or zones of bedrock may intersect a horizontal surface along the bedding dip direction (perpendicular to bedding strike) (e.g., Figure 4b). Any layer or zone that is penetrated by a borehole is not representative of the surrounding bedding to either side along the dip direction. Likewise, a sample taken from that layer is not representative of the surrounding bedrock. Despite these drawbacks, boreholes remain as an effective way to assess soil that overlies dipping bedrock, and to locate the soil/bedrock interface.

Trenches – Trenching is effective in areas underlain by steeply dipping bedrock because it exposes many near-surface bedrock layers for inspection and sampling (Figure 4b). It is especially useful for exposing discrete geologic features, such as bentonite beds, bedding-plane shears, or crosscutting shear planes that are typically unrecognizable in auger-drilled holes. The careful logging of a trench oriented along the dip direction may reveal major and minor lithologic zonations, and the composition, relative plasticity, and two-dimensional geometry of the different zones. It may reveal the presence of ground-water conduits and barriers, and the presence of other geologic discontinuities that may indicate prior episodes of near-surface heaving movement.

A trench log is extremely useful for developing the sampling program for the site investigation. It is critical to recover samples from the most common and representative layers as well as discrete and occasional layers, such as thin bentonite seams. Grab samples or hand-driven brass tubes are typically used to recover samples from trenches. Shear zones, which may also cause dramatic ground heave, cannot be sampled in a way that produces meaningful laboratory-test results (as the bedding is often similar on each side of the shear plane). However, such zones should be given full attention as part of the overall evaluation.

Laboratory Investigations – Samples taken from test holes or trenches are tested for a variety of engineering properties in a laboratory. Such evaluations are a required practice in many areas of Colorado where expansive soil or rock is anticipated. These tests commonly include natural moisture content (ASTM D 2216), natural dry density (ASTM D 2937), particle size (ASTM D 0422), and Atterberg limits (ASTM D 4318). The Atterberg limits – liquid limit, plastic limit, and plasticity index – are used to assess the plasticity of the sample. Swell-consolidation testing, also known as the Denver Swell Test (ASTM D 4546) is used to compute a sample’s swell potential and swelling pressure.

In Colorado, some other types of in-situ and laboratory methodologies are beginning to be used. Suction testing to evaluate swell characteristics is not widely done in everyday practice, although some geotechnical companies have adopted it to augment other tests. For examples, see papers in this volume by McOmber and Glater (2003) and Vessely and others (2003). In-situ and laboratory testing using reflectance spectroscopy to discern certain engineering characteristics is in its experimental stage (Goetz et al., 2002); this methodology may eventually be applicable for characterizing dipping bedrock in trench investigations.

JEFFERSON COUNTY DIPPING BEDROCK REGULATIONS

Jefferson County enacted amendments regarding expansive, steeply dipping bedrock to its land development and building regulations in April 1995. A map overlay zone, called the Designated Dipping Bedrock Area (DDBA), was created as an administrative tool to delineate where the new regulations are applicable (Jefferson County, 1995). The regulations contain minimum standards for site geological and geotechnical investigations, overlot grading operations, and the design of roadways, cuts and fills, foundation systems, drainage systems, utilities, and remedial construction.

Detailed geological and geotechnical investigations are necessary at the rezoning stage of planning to delineate areas where favorable, near-surface geological conditions occur (e.g., thick surficial soils or non- to low-swelling bedrock), versus areas where differential bedrock heave is likely. The regulations require a minimum buffer of 10 ft (3 m) of overburden (natural soil or engineered fill) beneath the anticipated level of the bottom of building foundations in areas of potentially heaving bedrock. Where insufficient natural overburden is lacking in these areas, deep sub-excavation (overexcavation) and fill replacement is used as a mitigation method. For more information about deep sub-excavation, see the paper in this volume by McOmber and Glater (2003).

The site investigations are done in two parts. First, exploratory borings are drilled at a density of one hole per 250,000 ft^2 (23,225 m^2) to depths of 35 ft (10.7 m), or to 25 ft (7.6 m) if bedrock is encountered). If no bedrock is encountered within 15 ft (4.6 m) depth beneath anticipated base-of-foundation levels, then those areas are exempted from further exploration requirements, and dipping bedrock mitigation is not needed.

If, however, the top of bedrock is encountered at shallower depths than this, more-detailed exploration must be done to evaluate the potential for differential bedrock heave. This second level of site investigation involves digging trenches and evaluating continuous, two-dimensional exposures of the near-surface bedrock. Cross sections (trench logs) are required as part of the geological report. The sections are to show subsurface bedrock relationships including the soil/bedrock interface; detailed bedrock stratigraphy; dip of beds; frequency and distribution of joints, faults, and discrete zones of highly expansive claystone or bentonite; and sample locations.

TRENCHING AND TRENCH LOGGING

The goals of trenching in steeply dipping bedrock include (1) identifying geological features that indicate the potential for differential heaving, (2) locating those features within three-dimensional space and relating those occurrences to the rest of the site, and (3) determining a sampling program that will show the engineering properties of the bedrock at critical locations. The results of the investigation are used to help determine whether deep sub-excavation is needed. The following section contains suggestions and guidelines for conducting a trenching investigation in this geological setting. For a more-detailed discussion of these items, see Noe and Dodson (1999). For other more-detailed discussions of trenching as applied to other areas of geotechnical practice, see Hatheway and Leighton (1979), Hatheway (1982), and McCalpin (1996).

Locating and Digging Trenches

In map view, layers or zones of steeply dipping bedrock form stripes that are oriented along bedrock strike (as in Figure 1). The most effective means of evaluating these different zones is to orient the trench along the dip direction, cutting perpendicularly across the different zones. This allows for the exact inclination angle of the bedding dip to be determined (as opposed to apparent bedding dips, which would be seen if the trench was cut obliquely across bedding).

The trench should be dug in the area of shallowest bedrock, as determined by previous borehole drilling results. Shallow trenches are desirable from a timing, cost, and safety standpoint. Ideally, the trench transect should cross an entire site, so that all bedrock zones are investigated and tested. (In this case, zone refers to a grouping of bedrock layers that can be distinguished from other flanking bedrock zones. A zone may be lithologically homogeneous or may have a characteristic suite of heterogeneous lithologies). This may require digging more than one trench if there are discontinuous areas of shallow bedrock. In this case, each trench should be dip-oriented, and they should overlap along strike at the ends.

Large trackhoe rigs are recommended for this type of trenching (Figure 5). Trackhoes have the advantage over smaller backhoes in that they can dig faster and deeper, and produce a wider trench. During the trenching operation, the project geologist and engineer should be present to determine the depth and cross-sectional shape of the trench (Figure 6). Trench sidewalls should be sloped or stepped according to OSHA safety regulations for trenches. Slot-type trenches with shoring supports have not been used, to the author's knowledge; however, it is possible this method could be used, provided that both safety and sidewall exposure and access are attained.

Figure 5. Trenching operation using a trackhoe.

 

Figure 6. Examples of trenches in steeply dipping bedrock. The stepped trench (left) was dug with a backhoe and is safe for entry by geologists. The unsupported single-slot trench (right), dug by a backhoe, is unsafe for entry or sampling until it is properly shored.

The targeted depth of the trench at any point along its length depends on the depth in which the top-of-bedrock contact is encountered; this depth may change markedly from point to point. Typically, this is the contact between the overlying soil unit (if present) and weathered, in-place bedrock. Trenches should be dug at least 3 to 4 ft (0.9 to 1.2) into the bedrock. This is necessary in order to view the true orientation of bedding planes, as near-surface bedrock layers are sometimes bent due to gravity flowage, and to identify shear-slip planes.

Before trenching begins, decisions should be made about which wall of the trench should be logged and on which side of the trench the spoil dirt will be piled. It may be advisable to log the shaded wall, as direct sunlight may obscure contacts by producing shadows. The shaded wall will dry out less quickly, making it easier to recover samples that have nearly in-situ properties.

Trench Logging

Field Supplies – The author has logged numerous trenches in expansive, steeply dipping bedrock, and has observed or assisted with the planning and logging of several others. A short list of supplies needed for trench logging includes the following items:

(1) Shovel, mattock, rock hammer, and garden trowel
(2) String or twine
(3) Bubble string level
(4) Plastic flagging tape
(5) Large carpenters nails
(6) Surveyor spray paint, day-glow orange
(7) Gridded paper, with 1-in (2.54 cm) internally subdivided grids
(8) Foam board, 2 x 3 ft (0.6 x 0.9 m) or larger
(9) Mechanical pencils and extra leads
(10) Erasers – perhaps the most important item!
(11) Colored pencils and pencil sharpener

(12) Masking tape
(13) Camera, with flash, and film
(14) Photographic scale/ruler
(15) Brunton field compass
(16) Portable lawn chair (for wide-base or benched trenches only)
(17) Plastic sampling bags and sampling equipment
(18) Permanent marker pens
(19) GSA rock color chart
(20) Hydrochloric acid, diluted to 10%
(21) Heavy work gloves
(22) Field notebook, waterproof
(23) Measuring tape, in both metric and English units

Personal field items such as food, water, sunscreen, rain gear, and appropriate clothing are needed as well. Safety gear such as a hard hat, protective glasses, and steel-toed boots are recommended.

On-Site Preparation – After being dug, the trench needs to be prepared for logging by cleaning one of the walls with a shovel or bladed hammer. The clay "scrape," smeared on the sidewall by the backhoe bucket, needs to be removed in order to see the in-situ features in the trench wall. Afterwards, a reference grid system should be set up using twine, nails, and a bubble string level. Coordinates should be labeled using flagging or spray paint.

Before detailed logging begins, it is worthwhile to look at the trench from a number of different vantages and distances, in order to get an overall feel for the features that have been exposed. Nails and flagging may be used to mark pertinent features. In particular, the soil/bedrock interface should be identified and marked along the trench wall (see "Geologic Features," below).

The trench should be photographed for documentation purposes after cleaning the walls and setting up the reference grid, but before logging or sampling begins. This will ensure that the trench wall is photographed in its most pristine condition. Overlapping photographs should be taken along the entire trench wall, from a common elevation and distance (if possible). More-detailed photographs should be taken of key geological features. Trench exposures, shales in particular, may quickly begin to deteriorate and ravel following exposure to sun, wind, and precipitation. This may have unexpectedly beneficial consequences, as otherwise similar-looking zones may weather differently due to differences in grain size, composition, moisture content, etc. If this occurs, it may be useful to re-photograph portions of the trench wall at a later time in order to document those effects.

The geologist should consider the amount of detail that is necessary to include in the log. Trench logs may be either subjective (i.e., an interpretive log that includes only the most pertinent features) or objective (i.e., an accurate portrayal of all features in the trench wall, both major and subtle, with little subjective interpretation). Subjective logging may be done relatively rapidly because all of the extraneous features have been omitted; the main drawback is that it is not conducive to alternate interpretations. Objective logging results in an archival record of the trench wall, with little subjective annotation; this requires a large amount of time to do.

Given the goals of trenching in dipping bedrock and the time constraints faced by most consultants, most of the logging that has been done to date in Colorado's dipping bedrock has been subjective. As a compromise, the author recommends that subjective logging should be used to portray all major features in the trench as accurately as possible. Other features (e.g., a highly fractured zone where it is not desirable to draw all of the fracture planes) should be annotated using numbered symbols on the log and numbered statements on the logging sheet.

Logging – Logging should be done on a gridded paper, taped to a foam board, using a 1:1 vertical to horizontal scale (i.e., no vertical exaggeration). First, the trench boundaries (floor, sides, and ground surface) and the reference grid should be drawn. This is followed by infilling the detail, sketching the major geological features. The author recommends using colored pencils to color-key different geologic features (e.g., bentonite beds, ironstone beds, concretions, shear planes, soil/bedrock interface), and other items such as the reference grid and sample locations. Logs from sloped trench walls may need to be rectified to a true vertical plane at a later time, after logging.

An alternative approach would be to create a photo mosaic of the trench, overlay the mosaic with frosted mylar, and log the trench directly on the mylar. Some practitioners use this approach for paleoseismicity studies (McCalpin, 1996). It is useful for trench exposures where there are varying sediment-clast sizes and fabrics, and for relatively short trenches (in the range of 100 ft or 30.5 m, or less). Although the process is time intensive, the result is an accurate trench log that is easier to defend in the future. To the author's knowledge, the photo-and-mylar logging method has not been used for trench investigations in expansive, steeply dipping bedrock in Colorado. Some potential drawbacks are that much of the bedrock in this area is fine grained and nearly optically homogeneous on photographs, and that some trenches may be too long for practical photographic logging (on the order of several hundreds of feet, or tens to hundreds of meters in length).

Regardless of the style and intent used in logging a particular trench, it is important to view the trench wall from a number of different vantages while doing the logging. A "far" view, from at least the top of the opposite trench wall, is necessary for recognizing the gross field relationships of the different lithologic units and discontinuities. A "medium" view from about 3 to 6 ft (1 to 2 m) away is needed for working out stratigraphic and structural complexities at certain locations along the trench wall. A "close" view from less than 1 foot (0.3 m) away is necessary for identifying bedding contacts, lithologic composition, basic engineering properties, and discontinuities, either along the trench wall or in hand samples. The final log should reflect these various views.

An example of a color-coded log that was done in the field, using a gridded paper approach, along an exposed arroyo wall, is shown in Figure 7. The log shows color-coded bedrock and soil units and other geological features, and contains a number of annotations that describe the different units. Note the bending of bentonite beds at the soil-rock interface due to gravity flowage (i.e., creep). A site-investigation trench would need to be dug deeper than this by several feet in order to assess the true dip of the bedding, as well as the linear (fault or shear?) feature that is marked by a significant change in bedding dip.

Geological Features

The following is a short summary of geological features that may be encountered in trenches dug in expansive, steeply dipping bedrock. These features should be logged, or at least annotated, as they may have bearing on whether differential heaving is possible.

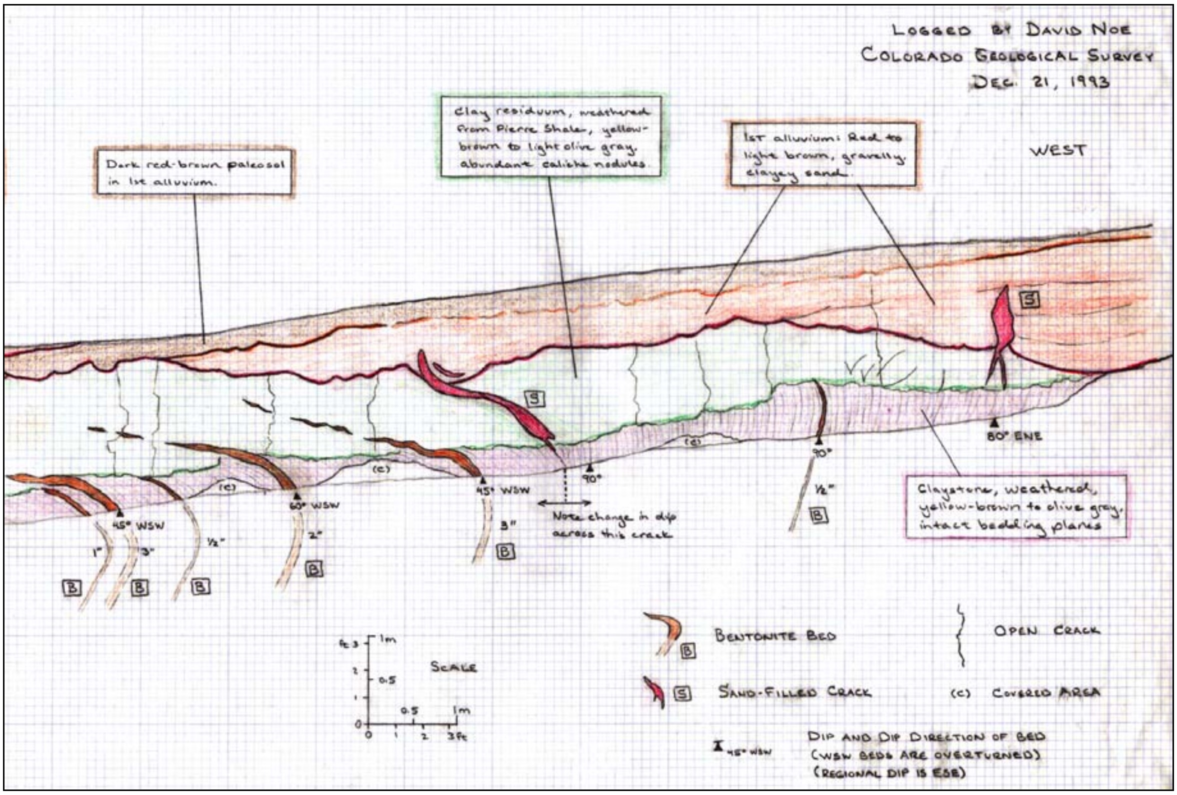

Figure 7. Part of a field log of an arroyo wall in Douglas County, showing color-coded geological features and annotations. Unpublished log by the author.

Bedrock layers or zones – The bedding contacts of major bedrock layers or zones should be shown on the log, especially if there appear to be changes in lithology and swell properties (Figure 8). The continuity of bedding along the trench walls is shown graphically by drawing the beds on the log sheet. Each zone should be annotated with information about rock type, color, grain size, sedimentary structures and, if possible, an interpretation of the unit's mineral composition, material properties, and Unified Soil Classification. In particular, all bentonite beds should be logged as distinct units, even if they are less than 12 inches or 30.5 cm thick (Figure 9); such layers may cause dramatic ground heaving because of their very high swell potentials.

Bedding dip – The angle of bedding dip is portrayed graphically, using the reference grid as a guide. If the trench is dug perpendicular to strike, then the true dip is exposed. Dip readings should be taken for certain beds where possible, using a Brunton compass, and annotated on the log sheet.

Fractures – The author has found most steeply dipping bedrock along Colorado's Front Range foothills to be highly fractured at shallow depths (Figure 10). It is probably not necessary to show all fracture planes in the bedrock; however, zones of especially highly fractured bedrock should be outlined and annotated on the log. Such fractures may be conduits for ground water, which may allow for relatively rapid and deep wetting of the bedrock.

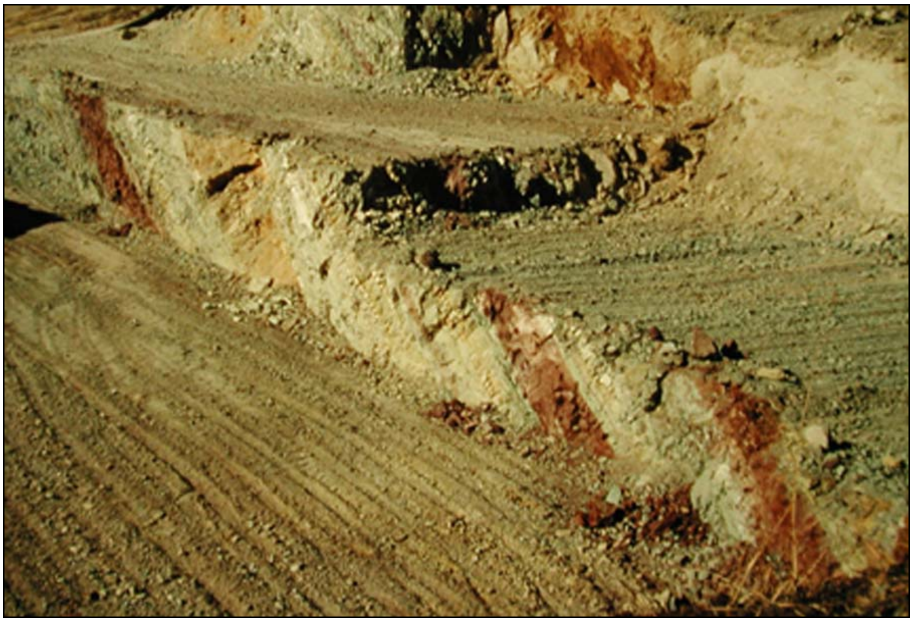

Figure 8. Steeply dipping beds of the Morrison Formation exposed in a basement excavation. Each of the bedding zones has a different composition and swell potential. From Noe and Dodson (1999).

Figure 9. Steeply dipping bentonite bed of the Pierre Shale. This 12-inch (30.5-m) thick bed has caused over a foot of vertical heaving in a nearby road (see Gill et al., 1996). From Noe and Dodson (1999).

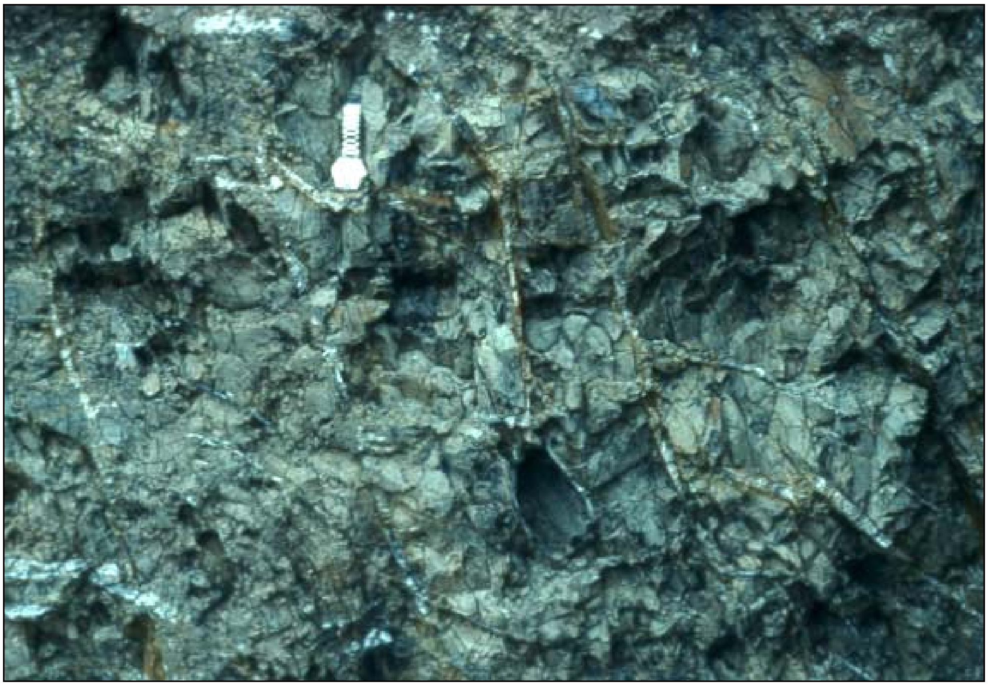

Figure 10. Highly fractured claystone of the Pierre Shale. These fractures contain veins of gypsum crystals, which are geochemical indicators of highly swelling bedrock. From Noe and Dodson (1999).

Shear planes – Shear planes are extremely important to recognize, as significant amounts of movement may occur between blocks of expansive claystone. Research by CGS shows that shearing movements of up to 3 ft (0.9 m) have already occurred along pre-existing fracture or bedding planes in the Pierre Shale, possibly as a consequence of differential, near-surface heave under natural conditions during the late Holocene. Movement along such surfaces may be re-initiated or significantly increased when the bedrock is exposed to abrupt increases of infiltrating water. Shear planes are often difficult to recognize, as they may resemble other fractures that cut across beds. They may be identified in trench walls and floors by finding offsets across the shear-plane surface in the adjacent bedded rock layers.

Bentonite beds in dipping bedrock in Colorado often have a sheared-looking fabric. It is possible that these beds have undergone internal shear movements, due to deep structural movements, because of their low shear strength. Particularly, when the Rocky Mountains were uplifted during late Cretaceous and early Tertiary time, soft bentonite beds may have served as slip surfaces for accommodating shortening as deeply buried, horizontally bedded shale was folded upward along the edge of the Front Range. Today, these beds most likely have low, residual shear strengths. This is a factor for consideration in slope-stability studies, but it may have an effect on dipping bedrock movements as well, as it may be possible for bentonite beds to undergo bedding-plane shear movements as well as expansive-clay swelling.

Accessory minerals – Gypsum is a chemical by-product of the leaching and weathering of pyrite-rich claystone. Its presence as a fracture-fill suggests that water has penetrated and chemically altered the claystone fabric in the past, and could do so again (Figure 10). Fibrous calcite, another weathering product, is almost exclusively associated with beds of bentonite. Observations by the author indicate that claystones and bentonites having the highest potential for expansion will often contain some secondary gypsum and calcite. These minerals, when encountered in a trench, should be sketched if part of a discrete bed, or annotated if found as fracture-fill.

Top of bedrock – The trench log should show the soil/bedrock interface, as well as all pertinent soil layers above the bedrock. The top-of-bedrock surface may be transitional, especially where a residual or colluvial soil overlies the bedrock. In such cases, there is usually "fugitive" material such as coarse quartz sand or crystalline rock fragments that have worked their way into the soil as part of the physical weathering process; the boundary is chosen as the lowest occurrence of fugitive sand grains or rock fragments. It may be identified visually with the help of a hand lens. It may also be identified by sound, by dragging a rock hammer downward numerous times along a cleaned exposure. The imbedded quartz grains in the soil will produce a clean, clicking sound. The claystone bedrock will not produce this sound, although imbedded ironstone fragments and gypsum bodies in the bedrock may produce a dull, thudding sound. Another means of locating the soil/bedrock interface is to identify the uppermost point where discrete beds such as bentonites lose their shape and fabric as they pass from the bedrock to the soil zone.

Moisture characteristics – All pertinent moisture characteristics should be annotated on the trench log, including the moistness of the different geologic units. Bentonite beds are often more moist than the surrounding bedrock; however, they may be capable of significant swelling and expansion nonetheless because of their higher plastic and liquid limits. The presence of groundwater seeps is important to annotate, as well as discontinuities that form boundaries between moister and drier bedrock zones. A heterogeneous ground-moisture system is an indicator of potential differential heave, as post-construction wetting will not infiltrate the bedrock evenly.

Samples

The trench logging should guide subsequent sampling of the trench. The object of the sampling is to characterize major geologic units or zones, and also to characterize the difference between adjacent units in terms of composition, moisture content, and other engineering properties. These differences may yield important insights into the potential for differential heaving of the bedrock. This type of sampling tends to be subjective in nature. However, one should attempt to take samples along the entire length of the trench in order to get a two-dimensional transect of bedrock properties along the dip direction. Sample locations should be marked on the trench log.

Report Presentations

When included in an engineering geologic report, trench logs should be reproduced and redrafted in a way that shows the internal geological features as accurately as possible. The use of patterned screens and other "cartoon" representations, in place of drafted features, is not advised. One of the biggest problems with including a trench log in a report is the odd size – some logs may be only a few inches (a few cm) tall while being several feet to tens of feet (several meters) long! One enterprising consultant has printed their trench logs on continuous-roll plotter paper, folded the paper numerous times, and bound the logs into the report.

A powerful means of showing the laboratory test results is to plot graphs that show the test results (Y axis) vs. the distance along the trench (X axis). This type of analysis of the test results emphasizes a two-dimensional spatial relevance. A series of single test-versus-distance data plots for tests such as moisture content, liquid limits, plasticity index, and suction may yield evidence of bedrock-zone relationships that may not be visual to the naked eye (Figure 11).

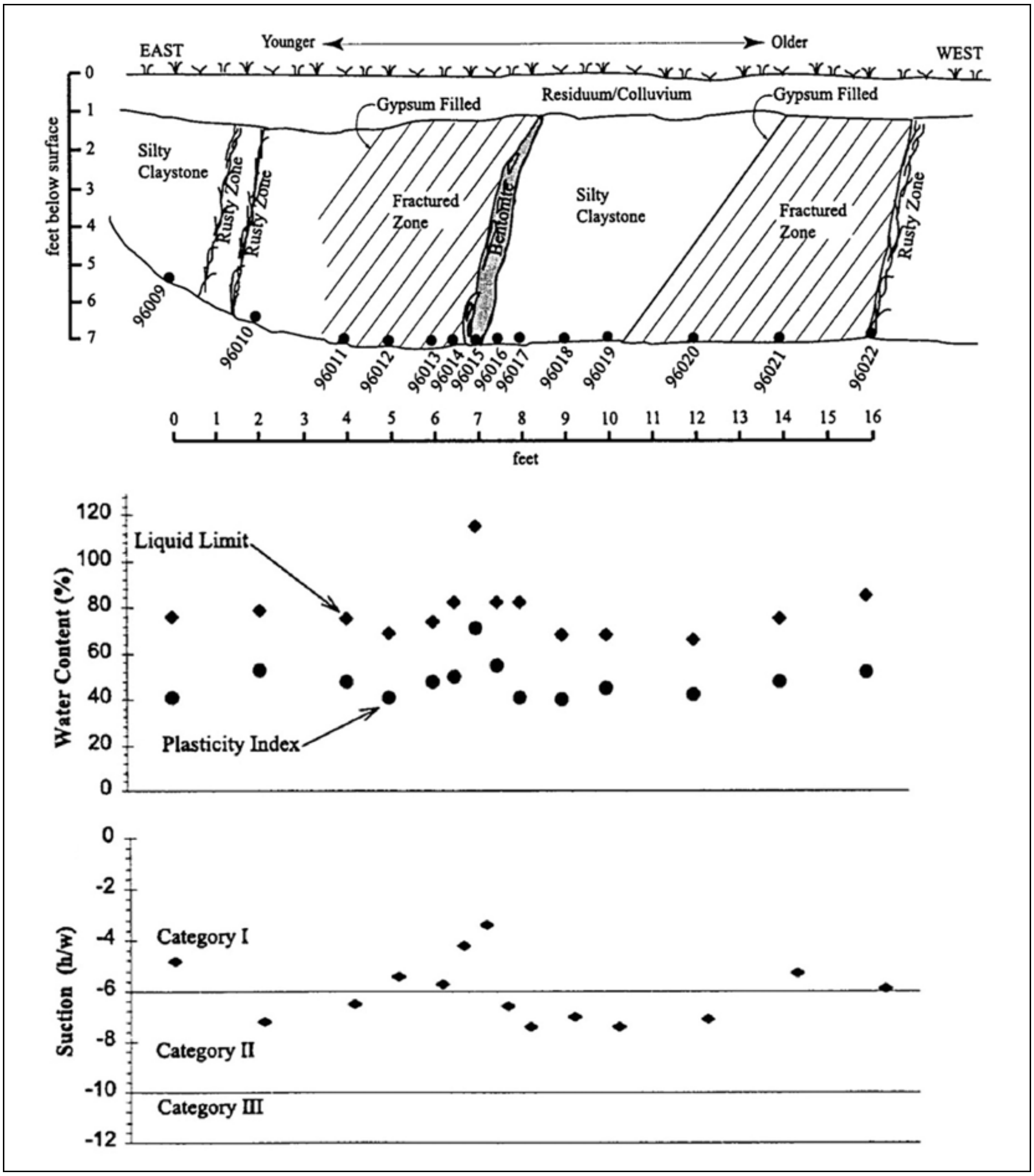

Figure 11. Example of a trench log from a geologic report, paired with graphs of sample test results along the trench transect. Trench log by the author; modified from Johnson (1998).

The results of a trenching investigation may be related or extrapolated to the rest of the site. For instance, highly expansive zones of bedrock encountered in a trench may be laterally continuous in the strike direction, particularly if the bedrock is a marine shale having relatively continuous bedding. Such zones, as extrapolated, would be seen in map view as "stripes" running across the site along strike. Other geological features, such as crosscutting shear zones and high moisture zones, may be more localized and are therefore less amenable to extrapolation.

RATING HEAVING BEDROCK HAZARDS

The main question to be answered by geologic and geotechnical investigations in steeply dipping bedrock is, "Is there evidence of a potential for differential ground heaving at this site?" If the answer is yes, then specific mitigation measures must be undertaken. The exact measures may depend on the scale of the project. For a single-family excavation, it may be possible to excavate in a way that considers the swell potential of each bedrock zone. This is a very tedious approach, however. In practice, and especially for larger projects such as subdivisions, the presence of potentially heaving bedrock at near-surface depths indicates that overexcavation and fill (deep sub-excavation) methods should be implemented to homogenize the subsurface materials.

Hazards from expansive, steeply dipping bedrock may be generally ranked according to the following descriptions prepared by Noe and Dodson (1999). In general, under the Jefferson County regulations, moderate- and high-ranked bedrock units would require overexcavation. There are low-ranked bedrock units, however, that could be exempted from the requirements by using careful drilling, trenching, and sample-testing procedures and documentation.

Low-Ranked Bedrock Units

Low-ranked units primarily consist of sandstones, non-swelling siltstones, limestones, or claystones with low swelling characteristics. Bentonite beds are absent or rare. Some units may contain minor interbeds of finer-grained material with low swelling potential. Damage is rarely observed. Atterberg Limit maximum values and ranges in values between adjacent bedrock zones are low, with typical liquid limit values ranging from non-plastic to around 40% and plasticity index values ranging from non-plastic to around 20%. These areas have a low potential for bedrock heave, and if bedrock heave did occur, a low amount of differential movement (less than 6 in or 15 cm vertical uplift) would be expected. However, trenching may be needed to define the boundaries of these units where they are in contact with potentially higher-swelling units.

Moderate-Ranked Bedrock Units

Moderate-ranked units contain both low- and high-swelling material. Bentonite is sometimes present. Bedding is continuous in some units, discontinuous in others. Damage is infrequently observed, although the magnitude of individual heave features may be low to moderate (as much as 6 to 12 in, or 15 to 30 cm of vertical uplift). Atterberg Limit values are variable, although the range between readings from different beds is usually moderate. The distribution of heave-prone areas and the severity of heaving may vary considerably. Trenching is critical in order to quantify variability and identify zones where heaving bedrock may be a problem. Overexcavation with fill replacement may be necessary as a mitigative measure over certain areas.

High-Ranked Bedrock Units

High-ranked units are primarily composed of very high-swelling claystone. Bentonite is common in some units and rare to absent in others. Damage has frequently been observed in these areas, and the magnitude of heaving may be low (less than 6 in or 15 cm vertical uplift) to severe (greater than 12 in or 30 cm vertical uplift). Atterberg limit values generally range from low to very high, with some liquid limit readings reaching 60% to over 120% and some plasticity index readings reaching 40% to over 70%, and high contrasts between adjacent strata are possible. Serious heaving bedrock problems will most likely be encountered within these areas unless localized geologic factors (e.g., thick overburden, high initial bedrock moisture content) are present to counteract the hazard. Overexcavation may be necessary in most cases unless otherwise indicated by trenching and other site-specific investigations.

There are several different types of factors that can influence the occurrence and severity of potentially heaving bedrock. These factors may be related to the composition of the bedrock itself, or they may be unrelated to bedrock composition. They may be natural or human-caused. Of all these factors, only bedrock dip and bedrock-unit boundaries can be readily predicted using published geologic maps and surficial geologic reconnaissance. The other factors, which are highly variable in their distribution, can only be assessed using data from site-specific subsurface drilling and trenching investigations.

CONCLUSIONS

1) Expansive, steeply dipping bedrock (also known as heaving bedrock) constitutes a distinct geological hazard in Colorado where the sedimentary bedrock layers are upturned and tilted. In such areas, the bedrock layers or blocks may swell unevenly to form parallel, linear heave features along the ground surface. Individual heave features may attain sizes as large as two feet high, several tens of feet wide, and several hundreds of feet long. This hazard has caused tens of millions of dollars in excess maintenance costs to homeowners, utility companies, counties or municipalities, and taxpayers.

2) The actual distribution and magnitude of heaving damage from expansive, steeply dipping bedrock is often variable on a local scale. The damage appears to be controlled by a number of interrelated, geological factors including bedrock sedimentology, structure, loading and unloading history, and moisture characteristics, and non-geological factors including foundation design, construction quality control, lawn irrigation, homeowner maintenance, and cumulative age of houses and other facilities.

3) The internal composition, geometry, and structure of expansive, steeply dipping bedrock allows for complex mechanisms of expansion and movement. These differences necessitate the use of distinct and hazard-specific site investigation approaches and techniques. Boreholes have a distinct disadvantage when the bedrock layers dip at angles of greater than 30 degrees because a sample taken from any particular layer is not representative of the surrounding bedrock. Trenching is useful for exposing discrete geological features, such as bentonite beds, bedding-plane shears, or crosscutting shear planes that are unrecognizable in auger-drilled holes.

4) Jefferson County enacted amendments regarding expansive, steeply dipping bedrock to its land development and building regulations in April 1995. The regulations contain minimum standards for site geological and geotechnical investigations, overlot grading operations, and the design of roadways, cuts and fills, foundation systems, drainage systems, utilities, and remedial construction. These regulations and minimum requirements have become the standard of practice for geologists and engineers in Colorado for this geological setting.

5) Trenches should be dug in the area of shallowest bedrock, and oriented along the dip direction, cutting perpendicularly across the different zones of steeply dipping bedrock. They should be dug at least 3-4 ft into the bedrock, in order to view the true orientation of bedding planes. The sidewalls should be sloped or stepped according to OSHA safety regulations for trenches.

6) The trench wall of interest should be cleaned, a reference grid system should be set up, and pertinent features should be flagged before detailed logging begins. Logging should be done on a gridded paper using a 1:1 horizontal to vertical scale (i.e., no vertical exaggeration). The author recommends that subjective logging should portray all major features in the trench as accurately as possible, and that other features (such as highly fractured zones) should be annotated using numbered statements on the logging sheet.

7) Geological features that have a bearing on the potential for differential heaving should be logged or annotated. These features include bedrock layers and zones, bedding dip, fractures, shear planes, accessory minerals such as gypsum and calcite, top-of-bedrock surface, and moisture characteristics. Sample locations should be logged as well. The object of sampling is to characterize major geologic units, and also to characterize the difference between adjacent units in terms of composition, moisture content, and other engineering properties.

8) Trench logs, when included in an engineering geologic report, should be reproduced in a way that shows the internal geological features as accurately as possible. The use of patterned screens and other "cartoon" representations, in place of drafted features, is not advised. A powerful means of showing the laboratory test results is to plot graphs of the results vs. the distance along the trench, to emphasize a two-dimensional spatial relevance. The results of a trenching investigation may be related or extrapolated to the rest of the site.

9) In general, hazards from expansive, steeply dipping bedrock may be ranked according to the geology and engineering properties. The delineation and ranking of bedrock zones that underlie a site requires an integrated drilling, trenching, and sampling program, and the close cooperation of the engineering geologist and the geotechnical engineer.

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MITIGATING HAZARD DUE TO EXPANSIVE, STEEPLY DIPPING BEDROCK AND FLAT-LYING SOIL AND BEDROCK THROUGH DEEP SUB-EXCAVATION

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Key Terms: expansive soil and bedrock, steeply dipping, heave, sub-excavation, ground modification, land development, case study

ABSTRACT

Uplift of the Rocky Mountains during Cretaceous and Cenozoic time has exposed tilted and fractured Paleozoic and Mesozoic sedimentary bedrock near the margins of the mountain front west of Denver, Colorado. Some of the steeply dipping bedrock, particularly the clay shale and claystone, is expansive (i.e., it has potential to swell when wetted). In areas where the bedrock is shallow, uneven wetting along bedding planes and fractures and variable swell within the layers can cause linear, differential heave features that are traceable over long distances. Deep sub-excavation has become the primary mitigation method in shallow bedrock areas. This method eliminates bedding planes, fractures and structure of the steeply dipping bedrock, and is commonly performed to about 13 to 20 ft below planned surface grades. The excavated material is mixed, moisture conditioned to above-optimum moisture and compacted, typically resulting in a low-swelling, relatively impervious clay fill that helps to limit downward seepage. Experience with projects completed since 1995 indicates success in mitigating the steeply dipping expansive bedrock hazard. Data from a site that has been monitored since 1998 is summarized as a case study. The use of the deep sub-excavation method has spread to other sites along the Colorado Front Range that are underlain by highly expansive soil and flat-lying bedrock. With this ground modification technique, builders, developers and their consulting geotechnical engineers and geologists are striving to reduce short-term damage to residences and light commercial construction and long-term maintenance for future owners.

INTRODUCTION

Deep sub-excavation is being widely used in the Denver area as a means to reduce risk of poor foundation and floor slab performance due to heave of expansive soil and bedrock. The method was first applied in the mid-1990s in expansive, steeply dipping bedrock in north-central Jefferson County, near Golden, Colorado.

As one travels east from the Front Range Uplift, across the western limb of the Denver Basin, progressively younger rocks outcrop or subcrop beneath the surficial soil deposits. The result, in the area within a few miles of the Front Range, is tipped bedrock layers that generally strike N 20 to 30° W and dip steeply at 30 to 80° (or more) to the east. In the Denver area, expansive, steeply dipping bedrock occurs in a ½ to 3 mi wide, northwesterly trending band in east-central

Jefferson County, continuing to the north into Boulder County, and to the south into Douglas County.

Over the past 30-plus years, experience has shown (Thompson, 1992a) that damage occurs about 3.5 to 7 times more frequently in residences constructed on shallow, steeply dipping expansive bedrock than residences located on bedrock in a nearly flat-lying orientation with similar measured swell (Figure 1).

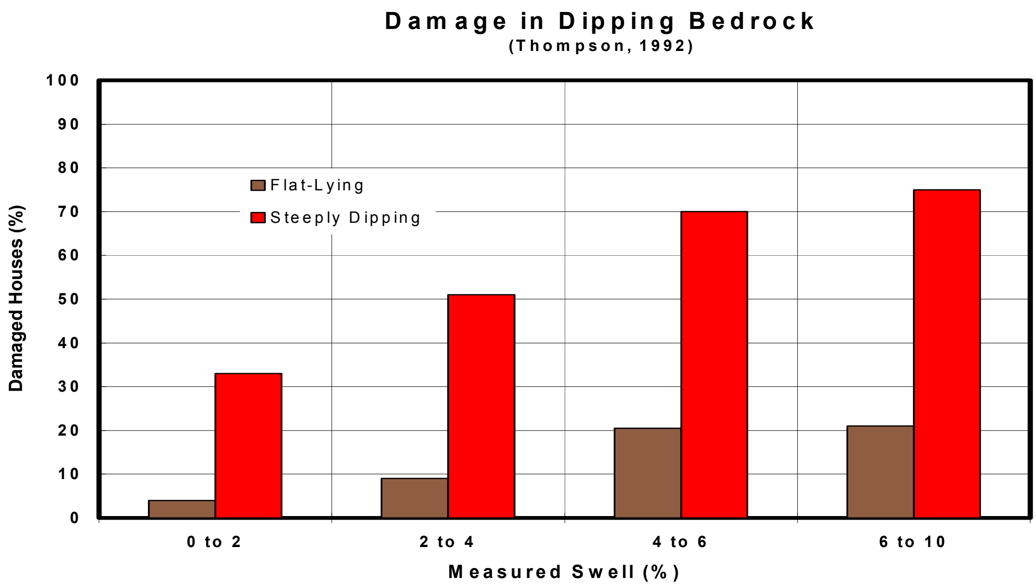

Figure 1. Swell vs. frequency of damage in expansive, steeply dipping bedrock (from Thompson, 1992a).

Commercial buildings and infrastructure such as roads, curb and gutter systems and wet utilities have also been affected by sharp, differential heave caused by uneven wetting of steeply dipping clay shales, most often within the Pierre Shale, but also in other Jurassic, Cretaceous and Tertiary sedimentary rocks such as the Morrison Formation, Graneros Shale and Laramie Formation. These heave features form lineations that can often be traced along strike over long distances (Weakly, 1989). Drilled pier foundations have commonly been used with success on expansive soil and bedrock sites in the Denver area. However, these foundations have been damaged by heave of the expansive, steeply dipping bedrock at a higher frequency than in flat-lying bedrock with similar swelling characteristics. In some cases the damage has included lateral forces caused by a horizontal component of heave of the steeply dipping bedrock.

A building boom occurred in the 1970s in southeastern Jefferson County, in the area of steeply dipping claystone bedrock generally west of Kipling Street, from Hampden Avenue south to about Chatfield Avenue. Forensic studies conducted since that time concluded the following:

- Where overburden depth exceeded about 8-10 ft below the basement or foundation, there was generally acceptable structural performance (Thompson, 1992b)

- Some residences on lots underlain by shallow Pierre Shale performed acceptably and some were rendered uninhabitable by differential movement. Trench observations, while often useful, do not always reveal geologic explanations for this behavior.

- Where entire subdivisions were studied and damaged homes were plotted on a map, they often lined up coincident with strike. With the indication that bedrock structure was involved, some practitioners surmised that deep, differential wetting of highly fractured zones was a cause.

- Recent work (Johnson, 1998) has shown the Pierre Shale (deposited in a deep ocean environment) contains layers with a high percentage of exchangeable sodium, which is related to gypsum content and results from the weathering and oxidation of abundant pyrite in some beds.

HISTORY OF DEEP SUB-EXCAVATION IN THE DENVER AREA

Jefferson County Task Force

As a result of the high frequency of foundation failures in areas underlain by steeply dipping expansive bedrock, lawsuits were filed against engineers, builders and developers in the 1980s. Many homes were damaged and some became uninhabitable. As a result, some engineers and geologists were not comfortable with continuing to recommend designs that appeared to have a high failure rate. Building officials were reluctant to allow development in areas that seemed doomed to failure. Developers were reluctant to build in high-risk areas.

It was clear that risk could be mitigated by avoidance. However, the area was also in a part of Jefferson County that was highly valued for schools, scenery and quality of life. Politicians, most notably Bill Schroeder, a state legislator, and Jan Rousselot, a member of the Jefferson County Planning Commission, were under siege from their constituents. Another residential building boom occurred in the early 1990s, bringing with it high development pressures and high rates of new damage. At the urging of Sen. Schroeder and Ms. Rousselot, a Task Force was formed in 1994 under the auspices of Jefferson County. Consulting engineers, geologists, homebuilders, developers, realtors, building officials, planners and municipal engineers participated in meetings over many months.

New Land Development Regulations

The Task Force meetings culminated in specific recommendations to Jefferson County officials in 1994 and 1995. The Task Force recommended detailed geological and geotechnical investigations and special mitigative designs for areas underlain by expansive, steeply dipping bedrock. Publication of the Designated Dipping Bedrock Area (DDBA) Overlay District Map by the Jefferson County GIS Department (Jefferson County, 1995a) and new Land Development Regulations (LDR) by Jefferson County occurred in 1995 (Jefferson County, 1995b). LDR Sections 9 and 10 specify requirements for detailed geologic and geotechnical investigations, and restrictions for development in the DDBA. For a discussion of DDBA site investigations and trenching guidelines, see the paper by Noe in this volume.

For structures within the DDBA, the LDR includes requirements for mitigative design that specifies at least 10 ft of fill or overburden be present below the foundation. For sites with basements, 16 to 20 ft of overburden from the finish grade to the bedrock is required. Other foundation alternatives may be considered pending County review. An Engineering Advisory Board was created to assist the County in this review. Because few sites in the DDBA contained at least 16 to 20 ft of low-swelling overburden that would allow use of footing foundations, deep sub-excavation became a standard of practice in the DDBA.

Early Deep Sub-Excavation Projects in the DDBA

Canyon Point – A parcel of land northwest of Golden, Colorado, north of State Highway 58 and west of State Highway 93, was developed using deep sub-excavation prior to the Task Force work. Subsurface exploration and trenching indicated the parcel was underlain by shallow, steeply dipping Pierre Shale. In 1993, CTL/Thompson, Inc. recommended deep sub-excavation for the project. The site was developed and site grading occurred during 1994-1995. To our knowledge, Canyon Point was the first deep sub-excavation site in Jefferson County. The developer has reported no foundation performance problems related to the sub-excavation.

Sunrise Creek - Shortly after development at Canyon Point started, Sunrise Creek, a site near South Simms Street and West Belleview Avenue in southeastern Jefferson County, was proposed for development. The geotechnical consultant (GTG/Fox Geotechnical Services, Ltd.) used extensive trenching and borings to evaluate subsurface conditions, which included steeply dipping bedrock from the Pierre Shale, Fox Hills Sandstone and Laramie Formation. These investigations were ongoing during the Task Force work. Development of the site, including deep sub-excavation, commenced in late 1995. The developer/builder has reported good foundation and basement floor slab performance, and streets have performed very well.

The Sanctuary at the Meadows - The Sanctuary, located north of Ken Caryl Avenue between South Simms Street and Kipling Parkway, was proposed for single-family residential development in 1995. Geologic and geotechnical studies were performed in 1995-1996 that included four trenches perpendicular to strike and six borings over the 40 acre parcel to supplement 15 borings drilled in 1994. The site is underlain by shallow Pierre Shale (Upper Shale Unit) and is located along strike with some of the most dramatic heave features in Jefferson County. The Sanctuary project is presented later in this paper as a case study.

DEEP SUB-EXCAVATION PRACTICE

Depth

The Land Development Regulations published by Jefferson County require at least 10 ft of overburden or fill beneath foundations in the DDBA. Some consultants have recommended 12 ft or more of sub-excavation with conditions of relatively higher swell or where school, commercial or multi-family construction is planned without basements. Basement construction typically adds 6 to 8 ft of necessary overburden thickness. Similar excavation depths are used for deep sub-excavation in areas outside the DDBA.

Jefferson County initially (i.e. 1995) required at least 5 ft of sub-excavation for all streets within the DDBA, and later amended the requirement to only include sites where claystone exists within 5 ft below the pavement. Experience has shown sub-excavation of 3 ft or more can improve pavement performance. Lime treatment is often used to stabilize the upper 8 to 12 in of pavement subgrade to enhance performance of flexible pavements on moist clay fill. Clays treated to optimum moisture content or above often have a relatively low modulus of subgrade reaction and can benefit from lime stabilization.

A review of current practice indicates deep sub-excavation depths are typically 15 to 20 ft below final grades. The depth of sub-excavation should be evaluated on a case-by-case basis considering the structures planned, geotechnical and geologic conditions, depth of potential wetting, depth of ground water and calculated potential heave. Conditions that may indicate decreasing the sub-excavation depth could include a thick, low-swelling or non-expansive layer such as sandy alluvium or sandstone below a target layer of relatively high-swelling soil or bedrock, or wetted soil and bedrock in areas of high groundwater.

Lateral Extent

Deep sub-excavation should extend below the entire building footprint. In the steeply dipping bedrock areas, lateral heave of bedrock adjacent to the excavation can affect foundations. Most consultants recommend the toe of the excavation extend at least 5 ft outside of the possible building footprint. Figure 2 shows a typical excavation profile. It is important to survey the bottom of the sub-excavated area so that building footprint can be compared with the documented extent of the sub-excavation. This was not done in some early projects and resulted in some structures being built partially in an un-excavated area, leading to differential heave.

Drainage

Where a sub-excavation project intersects the local groundwater flow or could result in damming of groundwater flow from up-gradient properties, an interceptor drain system should be considered to control the water. The homogeneous clay fill placed in deep sub-excavations can be much less permeable than the natural soil and bedrock, which can cause a damming effect, resulting in groundwater rise in up-gradient properties. Interceptor drains typically consist of a geo-composite drainage board placed on all or a portion of the sub-excavation face. A pipe is used to convey the water collected in the drain and discharge it to the ground surface or storm sewer. It is possible that a pumped lift station could be used to evacuate the drain system to the storm sewer or to a surface pond or drainage; however, the authors are unaware of such an installation to date.

Jefferson County requires installation of an underdrain system within streets to provide a conduit for connection to individual, perimeter foundation drains and to help control ground water after development due to irrigation of landscaping and precipitation. Underdrains consist of a network

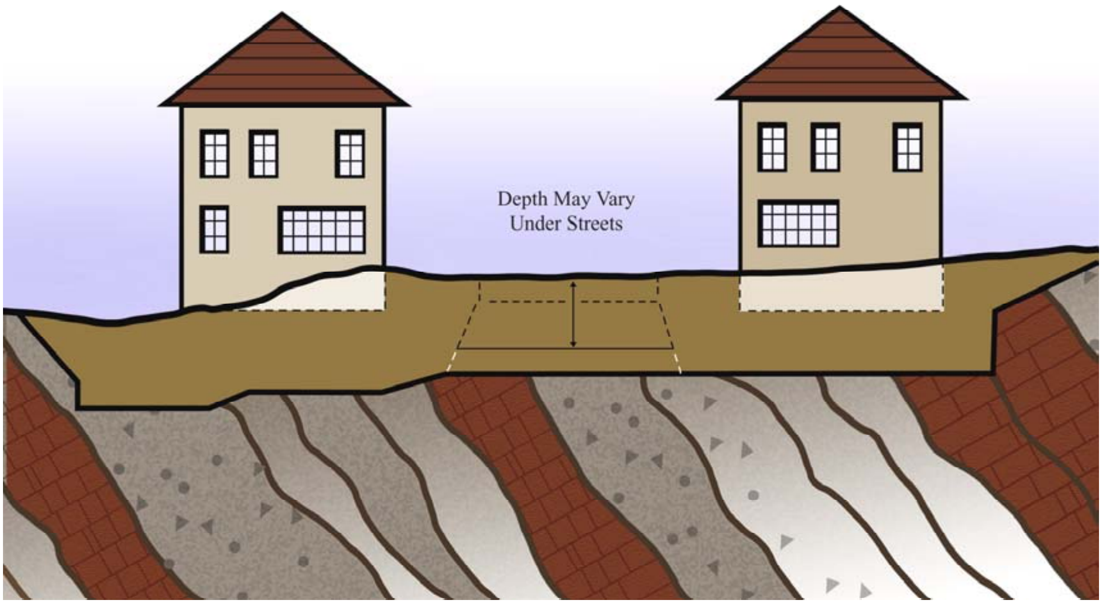

Figure 2. Typical deep sub-excavation profile. In Jefferson County, 10 ft of overburden soil or fill must separate foundations from the bedrock.

of pipe installed slightly below and offset from sanitary sewer mains (hence the term, "underdrains"). Some local jurisdictions and water and sanitation districts do not allow underdrains in the sewer trench.

Sub-Excavation Procedures

Most sites are sub-excavated in dry, high-swelling soil and bedrock, implying moisture contents less than optimum for the natural material, sometimes 6 to 8 percent or more below optimum. Often, the soil and bedrock materials have a blocky character formed by deep fissuring due to dessication and deformation. It is time consuming to process these materials into a broken-down, homogeneous, low-permeability, low-swelling fill. Compared to normal cut and fill grading projects, a higher proportion of processing equipment such as tractor-pulled disc harrows and large water wagons is used relative to the number of scrapers. Deep sub-excavation projects often have compaction performed by wheel rolling with scrapers rather than the self-propelled sheepsfoot compactors common to normal grading projects. Production is slower than normal grading jobs, mainly because of the increased processing necessary to bring the moisture content well above optimum and the difficulty of traversing the comparatively wet fill.

Some single-family residential sites have been sub-excavated on a single lot basis. For these small-scale projects, a front-end loader is often used to excavate, process and compact the material. Water is sometimes added by spraying with a hose. There is not enough room in the excavation to operate larger earthmoving equipment specifically designed to properly process soil materials. Therefore, there is a greater potential to inadequately process and moisture condition the material to reduce swell and compact the material properly to reduce potential settlement. In particular, it is difficult to compact the materials along the edges and in corners of the excavation. A higher proportion of problems with foundation and floor slab performance have been reported for such projects.

Testing during Construction

A goal of deep sub-excavation projects is the use of shallow, footing or pad foundations and slab-on-grade basement floors to separate the foundation from the native soil or bedrock below the fill. Most geotechnical engineering consulting firms in the Denver area recommend essentially full-time observation and compaction testing of the fill to help document the contractor's progress and methods. Additionally, some engineers obtain daily samples of the fill for swell testing during grading to help evaluate the effectiveness of the sub-excavation process. The samples are usually obtained by hand-driving a thin-walled (commonly 2.0 in OD) tube into the fill. Testing is performed in a one-dimensional swell-consolidation apparatus by wetting under an applied pressure of 500 or 1,000 pounds per square ft (psf).

A key component to the reduction in swell is moisture control in the compacted fill. Most projects are tested and evaluated using standard Proctor criteria (ASTM D698 or AASHTO T-99). Specified moisture typically ranges between optimum and 4 percent above optimum. Minimum compaction is typically 95 percent of ASTM D698. The authors' firm has implemented additional requirements for statistical evaluation of the process. For example, a minimum daily average test moisture content of 1.5 to 2 percent above optimum may be recommended.

Testing after Construction

Once grading is complete, additional subsurface exploration can be conducted by drilling and sampling to further evaluate the fill for structure support and swelling characteristics. For residential sites, most geotechnical engineers in Denver drill and sample one boring per lot when at least 3 lots are evaluated together. Samples are tested to evaluate moisture, density, swell, and other characteristics. Sampling with the modified California sampler (2.5 in OD, 2.0 in ID) commonly used in the Denver area can result in sample densification and hence higher swell test results when compared to thin-walled tube samples. This sample disturbance is not as noticeable in natural soil and bedrock samples.

Foundations - Experience shows that most samples from a fill controlled to an average of at least 1.5 to 2 percent over optimum moisture content, regardless of the plasticity of the parent material, will swell less than 2 percent when tested under a load of 1,000 psf. The Colorado Association of Geotechnical Engineers (CAGE, 1995) has defined swell of less than 2 percent as low when the sample is tested under a load of 1,000 psf. For conditions of low swell, local practice generally allows the use of shallow, footing type foundations, often with some required deadload pressure to help control differential movement. It is important to understand that the sub-excavation technique is used to control potential heave and differential movement, not eliminate movement. Foundation walls and grade beams should be designed to be comparatively stiff because movements will occur. Slab-on-grade basement floors are typically allowable on low-swell sites, provided the risk of heave and associated damage is acceptable to the builder/owner.

Drying of the Fill - The Denver area has a dry climate. Over the last several years, drought conditions have developed in the area. The lake evaporation rate, net of precipitation, is about 22 to 25 in, possibly more (PTI, 1996). The success of the deep sub-excavation process is predicated in part on retaining relatively high moisture content in the fill. If building construction proceeds shortly after grading, with associated covering of the ground surface with buildings, pavements, flatwork and irrigated landscape, experience indicates that drying does not occur. In such cases evaporation is limited by cover, and irrigation after construction and landscaping supplements precipitation. For sites that are exposed to the natural climate after grading, without full development and construction, some drying from the surface should be expected. Re-working of the fill may be necessary, particularly if a site sits vacant after grading for more than about 2 years.

CASE STUDY: THE SANCTUARY AT THE MEADOWS

Background

The Sanctuary at the Meadows is a 40-acre (+/-) site located along the north side of Ken Caryl Avenue between Kipling Parkway and Simms Street in the Designated Dipping Bedrock Area (DDBA) of Jefferson County, Colorado. An un-named tributary of Massey Draw forms the northern site boundary. The property slopes down from Ken Caryl Avenue to the north at about 8 to 25 percent.

The Sanctuary is underlain by the upper shale unit of the Pierre Shale. The unit includes moderately to highly plastic claystone bedrock. The bedrock dips at approximately 65° to the east-northeast, with a strike of about N 25° W. Preliminary investigation of the site (CTL/Thompson, 1996) indicated that the comparatively unweathered claystone was shallow below the south part of the site (less than 5 ft in depth), with a thicker clay or highly weathered bedrock surface layer near the drainage along the northern site boundary.

The performance of streets and houses along strike to the southeast of the Sanctuary indicated a high risk of damage to the surface improvements, buried utilities and residences planned at the site. Mapping of surface heave features (Weakly, 1989) and experience of the authors' firm with damaged residences to the south were combined to illustrate this risk, as shown on Figure 3. Some of the heave features within streets in the vicinity had created differential vertical movement of 6 to 18 in over distances of 40 ft or less. Most of the damaged residences had been constructed on straight-shaft, drilled pier foundations.

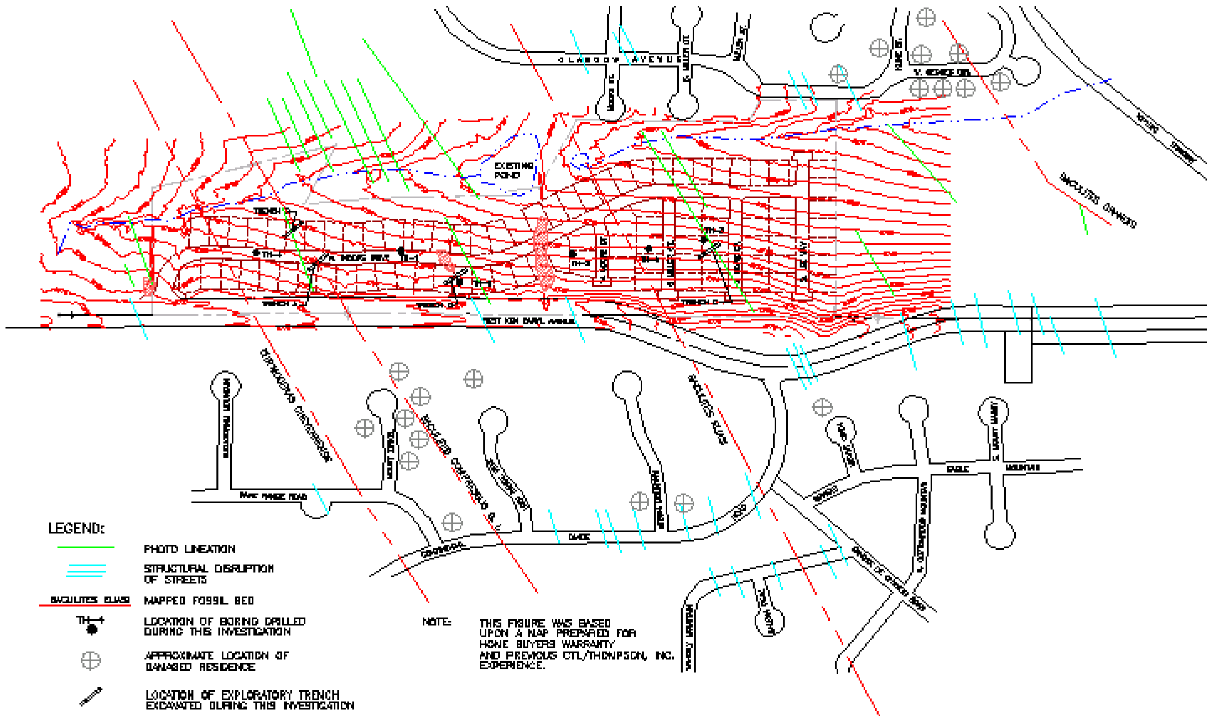

Figure 3. Evidence of geologic hazard in developed areas to the north and south of the Sanctuary, including damaged houses and linear heave features in streets (from CTL/Thompson, 1996).

Investigation Prior to Development

During the preliminary investigation by the authors' firm, six exploratory borings were drilled to a depth of 40 ft, and samples were obtained at 5-ft intervals by driving a modified California sampler (2.0 in I.D, 2.5 in O.D) into the bedrock. This sampler includes four, thin-walled brass tubes (1.935 in OD) that enclose the samples. Samples were tested for swell-consolidation using methods described by ASTM 4546 and local practice. The samples were loaded to pressures approximating the existing overburden pressure, wetted, and allowed to swell until no further movement was measured. The natural residual soils and claystone bedrock swelled about 7.6 to 8.9 percent at the 4-ft depth, decreasing to about 2 to 3 percent between 20 and 30 ft below ground surface. Samples remolded using standard Proctor effort swelled less than 3 percent at moisture contents more than 1 percent above optimum when tested under a 1,000 psf confining pressure. Liquid limits of the claystone materials ranged from 50 to 68 percent and plasticity indices ranged from 34 to 48.

A linear, marshy area with cattails was present on the slope below Ken Caryl Avenue prior to construction. Four exploratory trenches were excavated in an east-west orientation during studies in 1996 to examine the structure of the bedrock. One of these trenches was located above the marshy area. Seepage occurred into this trench from a fractured lense in the dipping bedrock, along strike with the marshy area. The estimated seepage rate was 75 gallons per hour. Moisture and density testing of the exposed claystone was performed in the trenches. The in-situ moisture content of the claystone varied between about 15 and 30 percent. Dry density varied between 91 and 114 pounds per cubic ft. Infiltration test results in the trenches varied between about 20 minutes per inch in highly fractured claystone and nearly 300 minutes per inch in more massive bedrock layers. Crystalline gypsum was observed in many of the bedrock fractures

Calculations of potential heave were performed based upon the results of the swell tests using methods described by Thompson (1997) and McOmber (2000). The calculations assumed that materials within 28 ft of the ground surface would contribute to potential heave, and indicated up to 12 in of potential heave.

Based upon the potential movement and risk indicated by performance of nearby structures, deep sub-excavation was considered to control the potential movement and differential movement. Finite element analysis was used to supplement empirical evidence (Thompson, 1992a) and help to evaluate how deep to sub-excavate. The finite element study was based upon input of a displacement boundary condition at the bedrock surface in contact with the base of a fill layer. The amount of bedrock displacement was selected to represent two heave features of 12 and 24 inches in height (assuming no sub-excavation), and was reduced with increasing depth of fill based upon the contribution to total ground surface heave of the bedrock removed and replaced by the fill. Figure 4 illustrates the results of the finite element analysis. For both heave features modeled, the reduction in differential movement at the ground surface was computed, and is plotted on Figure 5. The results indicate that differential ground surface movements would be reduced by about 90 percent with 16 to 18 ft of sub-excavation, to about 1 to 2.5 in. The analysis did not consider the reduction in risk of wetting of the bedrock below the fill as a function of low fill permeability and slower infiltration.

Dr. Delwyn G. Fredlund of the University of Saskatchewan (Fredlund, 1995a) was retained to peer review the studies by the authors' firm and perform independent analysis. Dr. Fredlund performed heave analysis using data collected by CTL/Thompson, Inc. and Kumar and Associates, Inc. from the Sanctuary site, as well as data collected by GTG-Fox Geotechnical Services from the Sunrise Creek site. Fredlund calculated maximum heave at the ground surface of 17.5 in based on a 40-ft depth of wetting. He determined that the optimum thickness of replaced, compacted fill was about 15 ft, resulting in a maximum predicted ground heave of about 5.2 in, or about a 70 percent reduction. Fredlund (1995b) conducted a finite element numerical analysis, which showed that the replaced fill buffered the amount of heave of the bedrock below the base of the replaced layer that will reflect up to the ground surface.

Site Development

The Sanctuary site was developed for 86 single-family residences in 1998. The majority of the site was sub-excavated to about 18 ft below proposed site grades. The excavated clay and claystone materials were used as fill. The fill was placed at high moisture to control potential swell. Street areas were treated in a similar manner to mitigate damage to buried utilities and pavements, and to increase the likelihood that surface water on the site would not seep through the fill and into the underlying bedrock. The excavation was designed to extend about 5 to 10 ft beyond the foundations at the perimeter of the site. The limits of the base of the excavation were

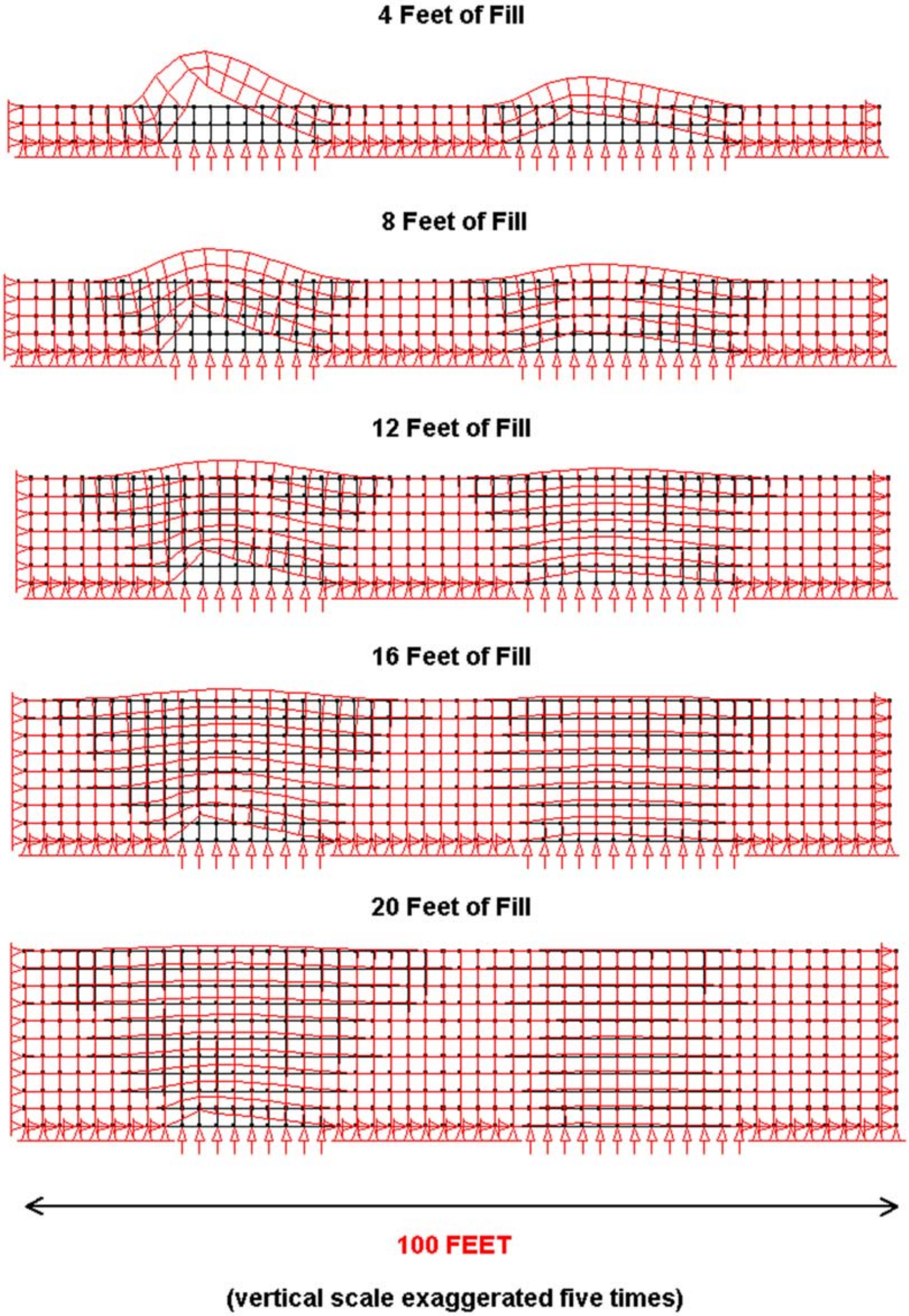

Figure 4. Finite element analysis results showing the buffering of two heave features (24 and 12 in high) by different amounts of overburden fill.

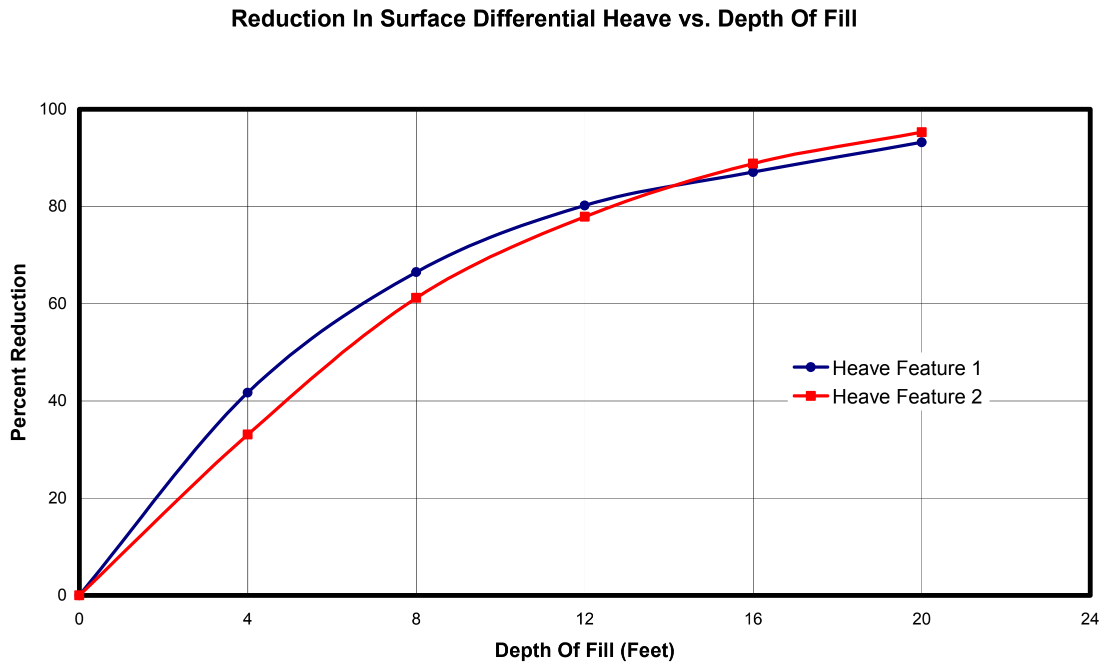

Figure 5. Reduction in predicted ground movement due to sub-excavation. Heave Feature 1 is 24 inches original height and Heave Feature 2 is 12 inches.

surveyed. An interceptor drain was constructed along the western portion of the south side of the excavation to capture observed seepage from the fractured bedrock.

Fill Moisture and Swelling Characteristics

Moisture and density tests were performed during site grading and sub-excavation using nuclear gauge (ASTM D2922 and D3017) and sand cone (ASTM D1556) methods. Samples of the fill were obtained by driving a thin-walled brass tube into the fill, and these samples were tested for swell in the laboratory.

Between August 1998 and January 1999, one exploratory boring was drilled on each of the 86 lots to depths of 15 to 35 ft and samples were obtained by driving a modified California sampler into the fill and underlying bedrock. It was observed that the California drive samples generally exhibited higher density and swell, and lower moisture content than the hand driven samples taken during site grading, and higher density and lower moisture content than the field density tests.

In May 2000, 25 additional test holes were drilled to depths of 18 to 30 ft to evaluate whether the fill had dried since placement, and to provide access for subsequent down-hole moisture measurements. Samples were obtained from these 25 holes by hydraulically pushing three-in Shelby tubes into the fill, in addition to driving modified California samplers. Samples obtained with Shelby tubes were initially prepared for swell testing by pushing a thin-walled brass tube (1.875 in ID) into the 3-in OD (2.85 in ID) Shelby samples. The laboratory swell test device was subsequently modified to allow direct testing of the Shelby tube samples.

The moisture and measured swell data for each of the sampling methods are shown on Figures 6 and 7, respectively. These data illustrate that the sampling method influences the measured swell of a moderate to highly plastic clay or claystone fill. In this case, the time lag between various sampling events may also influence the results.

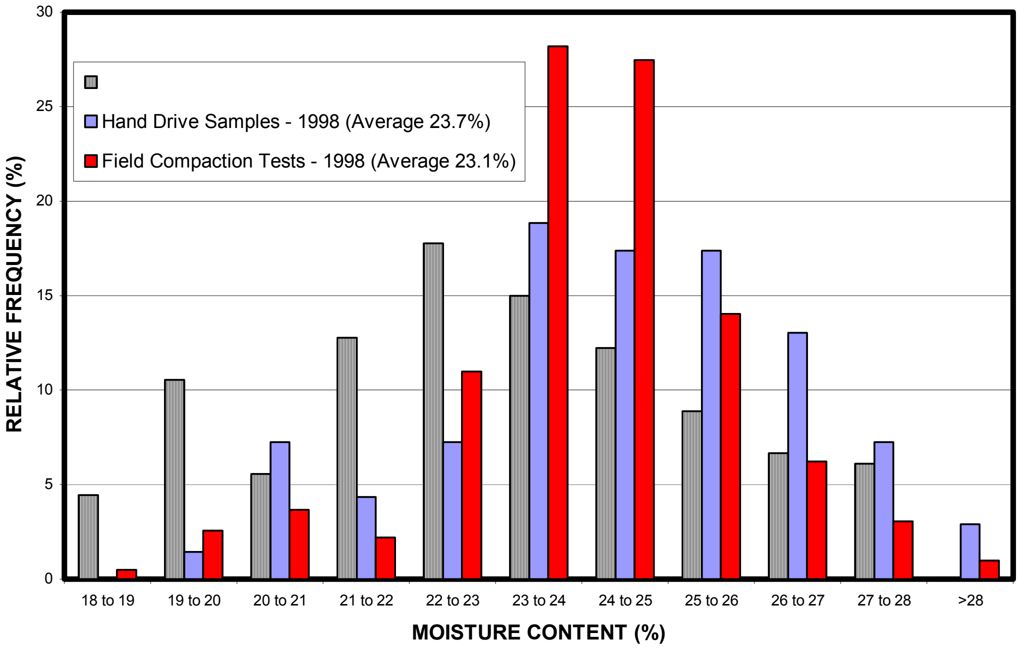

Figure 6. Sample moisture content for various sampling techniques.

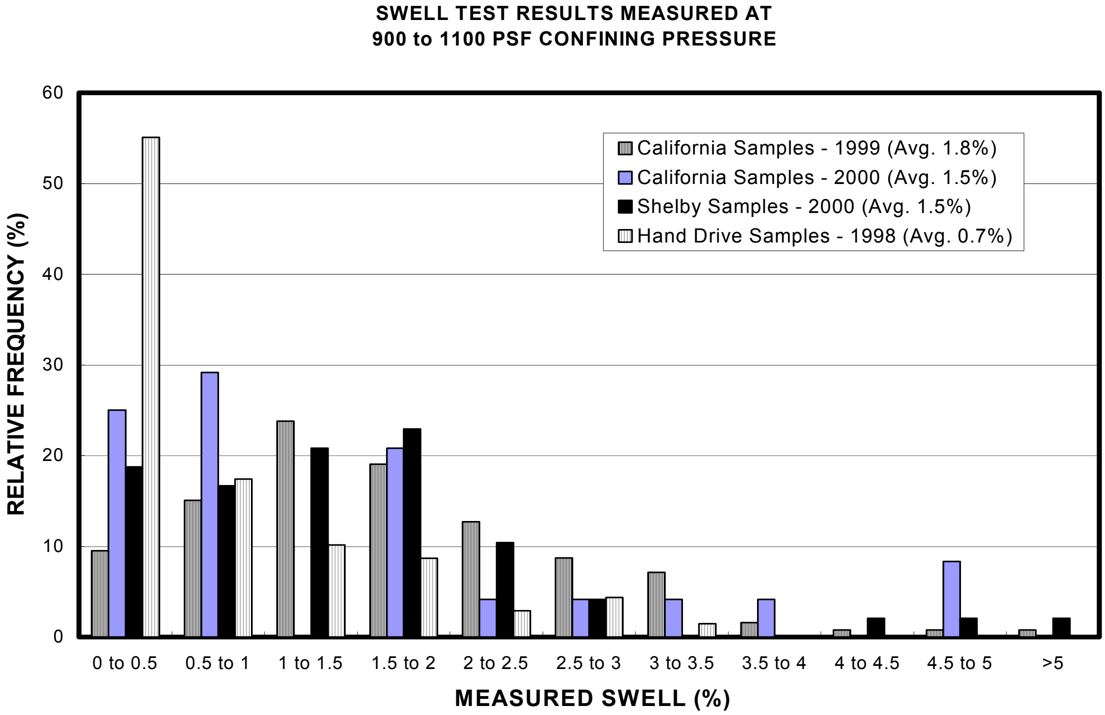

Figure 7. Measured swell for various sampling techniques.

Based upon numerous sub-excavation projects with data comparisons like those represented here, the following trends have been apparent:

1. Hand-driven, thin-walled brass tube samples obtained during fill placement generally exhibit density and moisture that is more consistent with field density testing than either Shelby tubes or California samples. The hand-driven samples also exhibit the lowest measured swell.

2. Shelby tube samples exhibit density and moisture that is more consistent with field testing than the California samples. The Shelby samples generally swell less than the California samples.

3. It appears that the California sampling process increases density, and therefore measured swell.

In May 2000, five 35-ft deep (+/-) cased observation wells were installed to provide access for downhole nuclear moisture-density monitoring. Three holes were drilled next to existing residences and two were drilled on lots where no residences or irrigation systems were present. The wells were drilled with a 2-in diameter, continuous-flight auger through the fill and into the bedrock. AW steel casing (2-in I.D., 2 3/8-in O.D.) was hydraulically pushed into each 2-in well. A sharp cutting shoe placed at the bottom of the casing "reamed" the fill and bedrock surface so that the casing had direct contact with the surrounding earth materials. The process created a compacted clay plug inside the casing. This plug was typically about 4 ft long, and effectively sealed the casing tip against ground water intrusion. Removable, water-tight plugs were installed at each casing surface collar, and access manholes were constructed flush with the ground surface and grouted to the casing collars.

Initial down-hole moisture readings were made in May 2000 and subsequent readings were made in November 2000, June 2001, and January, June and December 2002. The moisture contents from the initial down-hole measurements were compared to moisture contents from samples obtained from borings drilled within 2 ft of the observation wells. The nuclear probe moisture readings were 7 to 10 percent higher than the laboratory moisture measurements from soil samples at the same depths. The laboratory data were based upon oven-drying of soil samples. The down-hole measurements were determined from measurement of backscatter of neutrons radiating from an Americium source, and reflecting off hydrogen ions. In addition to presence of hydrogen within soil minerals, presence of a thin water film on the steel casing may influence the nuclear gauge measurements.

Given the differences in measured moisture content, further analysis focused on how down-hole moisture readings changed from the initial readings in May 2000 as an indicator of potential drying of the fill. The measured change in down-hole moisture content for two of the five monitoring wells is plotted on Figures 8 (Well #1) and 9 (Well #3). Well #1 is located next to an existing residence in a landscaped area near irrigated lawn. Well #3 is located on a lot where no residence is present, and no irrigation source is nearby.

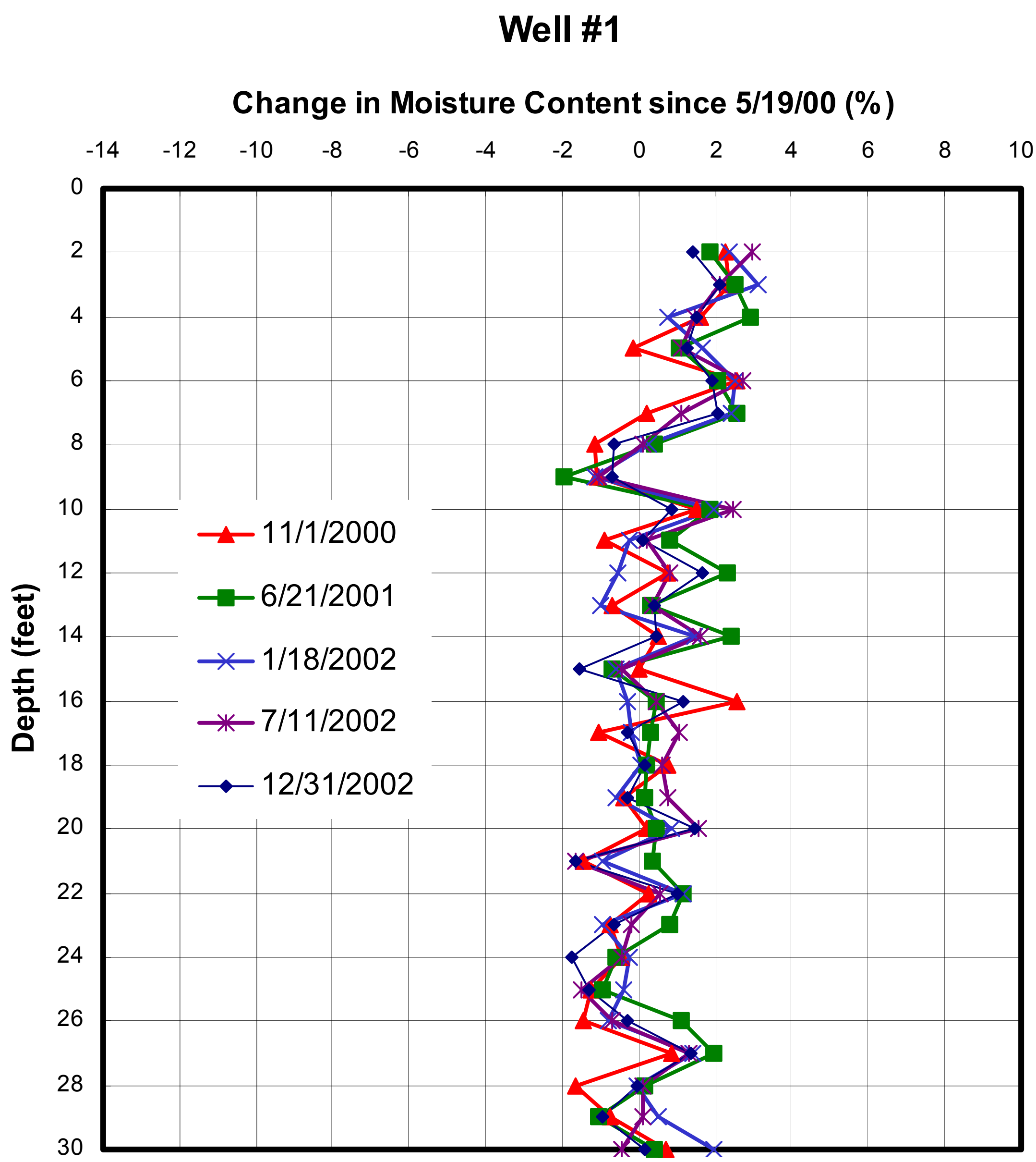

Figure 8. Changes in moisture – Well #1 (adjacent to landscaped area)

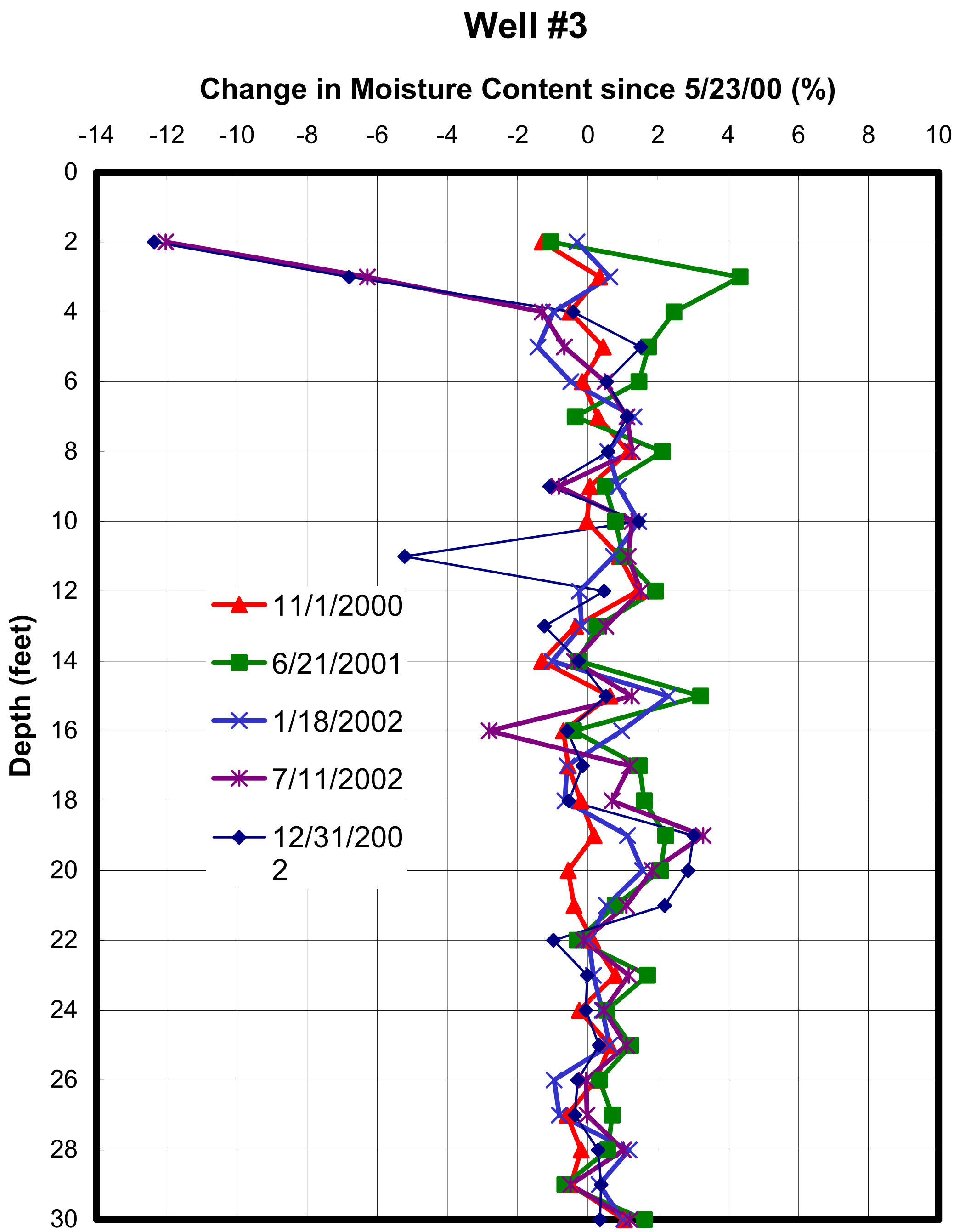

Figure 9. Changes in moisture – Well #3 (vacant lot)

The following observations and inferences are drawn from the data:

1. The average change in moisture for all depths in all five holes between the initial measurement and November 2000 measurements was 0.04 percent. The standard deviation was 0.9 percent moisture.

2. There is an indication of additional wetting of the fill within 3 to 7 ft of the ground surface in two of three wells located near existing residences and adjacent to irrigated areas over the 31 months of monitoring.

3. There is no significant evidence of drying of the fill in 3 wells located near existing residences and adjacent to irrigated areas (example: Well #1, Figure 8).

4. There is evidence of drying of the fill to a depth of about 4 ft in the July and December 2002 measurements from two wells on vacant lots (example: Well #3, Figure 9). Drought conditions have occurred in eastern Colorado since 2001. The data suggest that near-surface drying has occurred as a result.

The authors believe that the impacts of drying of the fill prior to construction can be mitigated by monitoring moisture conditions in foundation excavations, and either removing the dry materials or re-establishing moisture through re-processing or moisture injection. The measurements to date do not indicate drying of the fill around a completed residence.

Monitoring of Ground Movements

Ground movement (settlement or heave) at the ground surface at The Sanctuary site has been monitored using surveying methods. In May 2000, a deep benchmark was installed at the west end of the site. The elevation of the benchmark and the top, back of curb at each property line along the north side of the east-west street that crosses the site have been surveyed at approximate 6-month intervals since July, 2000. The depth of sub-excavation below the monitored curb area is on the order of 18 ft. The data indicate very little movement has occurred, as indicated in Table 1.

Table 1. Summary of survey measurements of curb, the Sanctuary at the Meadows.

Survey Date	Maximum Apparent Heave (inches)*	Maximum Apparent Settlement (inches)*	Average Apparent Curb Movement Since Initial (inches)*	Comments
July 12, 2000				Initial Survey
January 5, 2001	0.5	-0.8	0.00	
July 10, 2001	0.7	-0.7	0.04	
January 10, 2002	0.6	-1.4	0.07	
August 8, 2002	0.6	-1.7	0.21	

* Note: Calculated values are shown to 0.1 to 0.01-inch precision for illustration. The values are based on survey measurements to the nearest 0.01-foot. Positive values indicate heave and negative values settlement.

GROUND MODIFICATION OUTSIDE DIPPING BEDROCK AREAS

The use of the deep sub-excavation method has also spread to other sites along the Colorado Front Range that are underlain by highly expansive soil and bedrock. Many sites currently available for development in the Denver area present risk of damaging movement for shallow foundations and slab-on-grade floors. Ground modification such as deep sub-excavation can reduce risk of construction on these sites and often will allow use of shallow foundations and slab-on-grade floors. Similar depths of treatment to those used for DDBA sites are typically chosen. With this ground modification technique, builders, developers and their consulting geologists and geotechnical engineers are striving to reduce short-term damage to residences and commercial construction and long-term maintenance for future owners.

SUMMARY AND CONCLUSION

Deep sub-excavation has become the primary method for mitigating the impact of expansive, steeply dipping bedrock. Use of deep sub-excavation has spread to other sites along the Colorado Front Range where high-swelling soil and relatively flat-lying bedrock occur and developers choose to perform ground modification. The technique typically allows use of shallow foundations and slab-on-grade floors. Reported evidence shows that where the technique has been properly applied, the performance of foundations, flatwork, roads and underground utilities has improved compared to sites where sub-excavation was not performed. Monitoring of one site has demonstrated that fill materials have not dried significantly in areas where residences were built and landscaping is installed and irrigated. Some drying of the top 4 ft of fill occurred in drought conditions on lots where no construction has occurred. Survey measurements of curb movement indicate that sub-excavation has controlled ground movements during the first 4 years after site development.

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CASE STUDY OF THE EFFECT OF TRANSIENT WETTING FOR STRUCTURES ON EXPANSIVE SOIL AND BEDROCK

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Key Terms: expansive soil, swelling soil, depth of wetting, soil suction

ABSTRACT

Local practice and standard of care dictates that structures on expansive materials be designed to resist or accommodate swell due to expansive soil and bedrock. The local standard of care was developed based on depth of wetting assumptions. A depth of wetting in the subsurface is estimated based on the results of a geotechnical investigation and the local practice experience. Subsurface wetting events associated with excess irrigation or plumbing leaks are not typically considered during the design phase because of the increase in construction cost that would result when designing for low probability events. However, when a transient wetting condition occurs, it may be necessary to determine the effect of excess water on foundations and concrete slabs for insurance settlements and remedial design. This paper reviews three examples of transient wetting events that occurred below residential and commercial structures in the Denver metropolitan area. It discusses the effect of excess water on the building performance and the use of suction testing and moisture content analysis in determining the depth of transient wetting and in estimating the remaining potential magnitude of heave. The data suggest transient wetting may increase the rate of swelling, but not the magnitude of heave relative to long-term values.

INTRODUCTION

Subsurface Wetting Terminology Associated with Expansive Soil and Bedrock

Expansive soil and bedrock are defined as materials that have a potential to swell due to increases in moisture. The same materials also have a shrink potential when moisture contents decrease. The swell potential is due to a molecular-level force that attracts a relatively large volume of water to the clay particles, where an ionic substitution occurs during hydration.

The water content in the upper few meters of the subsurface is influenced by climatic effects, and this is commonly termed the “zone of seasonal fluctuation” or the “active zone” (Nelson and Miller, 1992). Additionally, the term “depth of wetting” is frequently used when investigating developed sites where the effects of construction and irrigation contribute to increased subsurface moisture contents. Figure 1 illustrates common subsurface moisture profiles.

In order to develop heave estimates for design, assumptions for the depth of wetting need to be established. As discussed previously, post-construction heave is likely to occur on developed sites

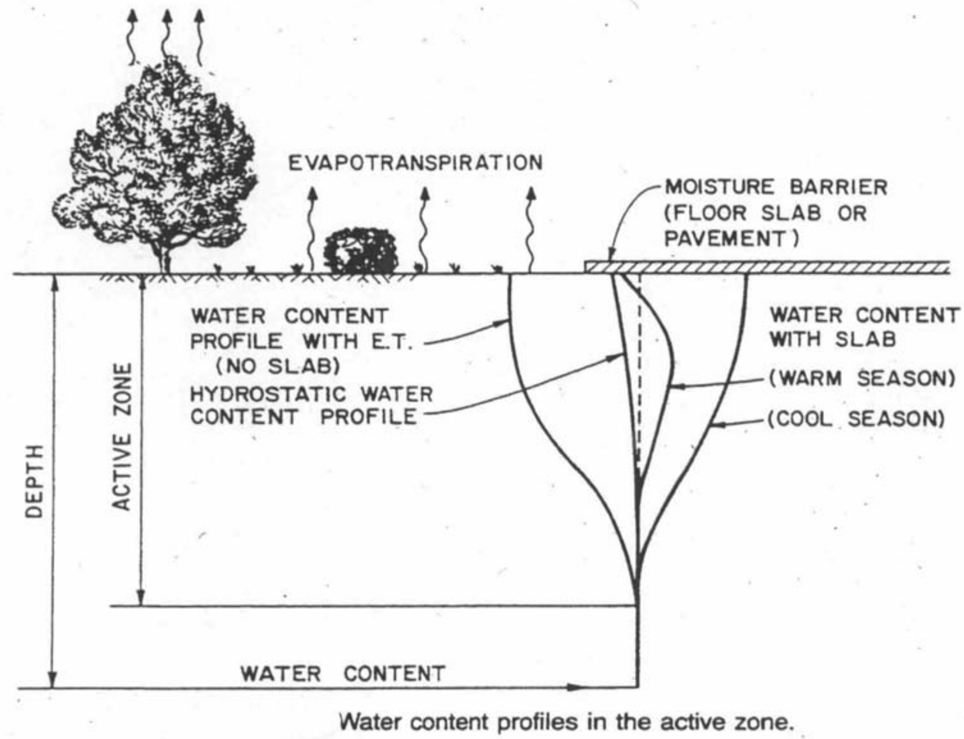

Figure 1. Active zone profiles (from Nelson and Miller, 1992).

with expansive material because of the high probability of subsurface moisture increase. In the greater Denver area, McOmber and Thompson (2000) compared the pre- and post-construction subsurface moisture data (suction) from residential areas within 12 years of construction. The authors suggest that an active zone of wetting occurs to about 15 to 20 ft (4.6 to 6.1 m) below the ground surface in residential areas due to landscape irrigation and impermeable pavements and structures. The paper concludes that seasonal variation of wetting is significantly reduced after construction, and that wetted soil and bedrock in these areas does not experience significant drying after construction. The presented data indicate that cumulative heave in soil and bedrock within the active zone decreases with depth because there is generally less variation in the post-construction suction potential, and therefore less potential for heave. Also, heave predictions may be overestimated for deeper materials within the active zone when using conventional heave estimation techniques (i.e., translation of percent swell).

Soil Suction

Expansive soil and bedrock does not need to be saturated for expansion to occur. An attractive force within soil and bedrock facilitates unsaturated moisture movement, and is termed suction potential. McKeen (1992) indicates that soil may be characterized by its total potential or suction. The suction consists of osmotic and matric components. Osmotic suction is a result of forces acting on water molecules, and is based on the chemical activity of the soil due to soluble salts in the soil water. To generalize, a chemical concentration variation between particles and water in soil or bedrock results in a pressure differential that is termed osmotic suction. Matric suction is the difference between the pore air pressure and the pore water pressure. Fredlund (1979) discusses the nature of the air-water interface and demonstrates that the interface can be considered as a membrane representing a distinct phase in soil.

Soil suction is commonly expressed in the pF unit, which is defined in American Society of Testing and Materials (ASTM) Method D5298 as "a unit of negative pressure expressed as the logarithm to the base ten of the height in centimeters that a column of water will rise by capillary action or negative gauge pressure (Mg/m^2)." The pF unit is approximately equal to three plus the base ten logarithm of the negative pressure in atmospheres. McOmber and Thompson (2000) states typical values of soil suction in the Denver area range from 3.5-to-4.5 pF. Low pF values indicate relatively low suction, corresponding to wet conditions and lower swell potentials. High pF values indicate relatively high suction, corresponding to drier conditions and higher swell potentials.

In geotechnical applications, the absorbed cations are generally fully hydrated and therefore osmotic forces are relatively constant. The changes in total suction values are primarily due to variation in matric suction. In general, the drier the soil or bedrock, the higher the pF value. As a specimen is wetted, the suction becomes a smaller value. The measurement of soil suction values in soil and bedrock can be used to estimate zones of moisture variation when designing structures on expansive soil.

Review of Denver Geotechnical Practice

Geotechnical professionals in the Denver Region must evaluate the risk associated with expansive soils and bedrock, and the owner/client must determine the acceptable level of risk for the proposed construction. The geotechnical practice in the Denver area uses a relative scale to evaluate swelling potentials. An index test is performed whereby the sample is wetted under a surcharge pressure, typically 1,000 psf (48 kPa), and the measured swell is classified as low, moderate, high, or very high. Table 1 presents the relative slab risk classification criteria for the percentage of expansion based on initial sample height at the indicated surcharge pressure for index testing.

Table 1. Risk Category and Swell Ranges

Risk Category	Percent Swell Under A 1,000 PSF (48 kPa) Surcharge Pressure
Low	0 - <2
Moderate	2 - <4
High	4 - <6
Very High	Greater than 6

Source: CAGE (1996)

The relative classification can be correlated to potential slab damage as follows:

Low - minor slab cracking, differential movement, and heave;
Moderate - slab cracking and movement, partial framing void closure; and

High to Very High – large scale slab cracking and differential movement, closed voids.

These effects are based on monitoring and observations by several firms in the Denver metropolitan area and are not limited to the relative classification. More or less damage can occur in all classifications because of the uncertainty in investigation and soil sampling, inherent to determining geologic and soil conditions.

Post-construction increases in soil moisture content greatly affect structures on expansive soil and bedrock. Slabs, pavements, and structures significantly reduce evaporation of soil moisture, and the degree of saturation will increase below these moisture barriers. Typically, irrigated landscaping also will increase soil moisture above the pre-construction moisture content. Therefore, post-construction heave is likely to occur at sites with expansive soils because of the high probability of subsurface moisture increase. Poor maintenance and design, such as negative slopes adjacent to foundation walls and irrigated landscaping adjacent to the foundation, will also increase the risk for expansive soil and bedrock damage.

The prediction of heave in the Denver area is a subjective process. The current local practice and standard of care is to use the measured swell percent obtained from one-dimensional swell/consolidation tests on representative samples and then estimate the resulting field heave for the given material. The swell test values are typically reduced using an adjustment factor of 0.7 to predict actual heave. The adjustment addresses the influence of restraining lateral swell in the test method and the more complete flooding in the laboratory test. Additional data, such as soil suction values, can be used for heave estimation and determination of the depth of wetting.

SITE DESCRIPTIONS

The observation of transient wetting events on three sites is reviewed in this section. Because of the presence of pending insurance settlements, the specific locations of each of the three case-study sites are kept anonymous for this paper. The authors do not believe this detracts from the conclusions of the case studies, because the sites are all located in similar regional geologic and climatic settings. The three sites are located in areas with relatively flat lying bedrock stratigraphy.

Residential Structure 1

The residence was constructed approximately four years ago (c.a. 1999) and occupied and landscaped at about the same time. The structure is a two-story, single family residence with a 10-ft (3.0-m) deep basement and a structurally supported floor beneath approximately 70 percent of the residence. The remaining portion of the residence was constructed over a 3-ft (0.9-mr) deep a crawl space.

In March 2002, flooding of the interior crawl space was noticed. The source of the water was located on the western side of the residence where the water supply line enters (Figure 2). The pipe was repaired, and efforts were taken to remove free water and dry the exposed soil over a three-week period. The water volume discharged into the foundation soils is uncertain; however, based on anecdotal evidence, approximately 1.5 ft (0.5 m) of free water was present in the

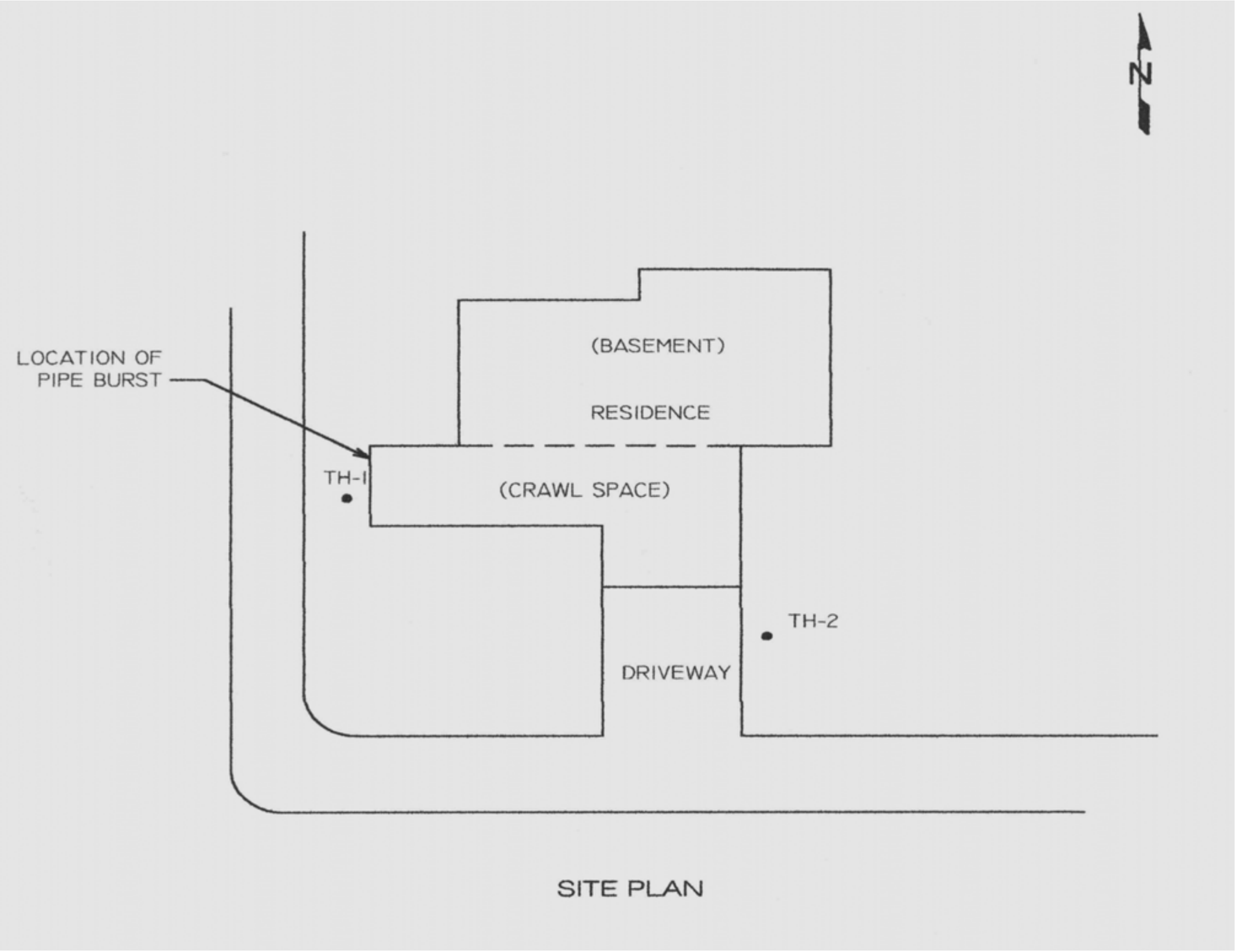

Figure 2. Site map for Residential Structure 1.

crawl space. In most areas, the void spaces below foundation walls and structural beams closed due to heave.

The original, pre-construction geotechnical investigation for the property encountered 16 ft (4.9 m) of sandy clay underlain by medium hard to hard claystone bedrock. Drilled piers, bottomed in bedrock with a minimum length of 25 ft (7.6 m) for basement piers and 29 ft (8.8 m) for non-basement piers, were recommended, along with a void of eight in (20 cm) beneath the grade beam. The swell potential for this particular lot was rated as very high per the local practice guidelines.

The water from the flooding event was transient in nature. Free-standing water was contained within the crawl space for an unknown length of time, facilitating wetting of the underlying clayey fill soils. When expansive clay is wetted, it absorbs water and expands. This expansion typically closes shrinkage cracks in the soil structure, and moisture migration via actual water flow is limited. Once the water was pumped out, very moist or wet conditions with limited evaporation existed in the crawl space. The moisture from the wet surface layer in the flooded areas is then migrated away from the flooded areas in three dimensions in response to soil suction. Initially, saturated flow may have occurred in the soil adjacent to the water; however, unsaturated flow likely occurred with increasing distance from the source for a time after the event. In general, the entire property is subject to limited moisture evaporation because of regular irrigation during the summer evaporation period and subsurface moisture conditions remain elevated.

It is not believed that the flood water migrated downward to the ground water level observed during drilling operations. Suction test data collected after pipe rupture indicate additional wetting in the near-surface clay fill soils, but not in the materials deeper than approximately 15 to 17 ft (4.6 to 5.2 m) below grade (Figure 3, profile TH-2). The claystone bedrock that was encountered at approximately 14 to 19 ft (4.3 to 5.8 m) below grade was more resistant to water infiltration than the clay fill. Based on data in Domenico and Shwartz (1990), the average of representative hydraulic conductivity values for clay is about 0.2 ft/year while the average for a fine grained bedrock is 0.1 ft/year. Therefore, free water may collect in the clay fill above the less-permeable claystone bedrock.

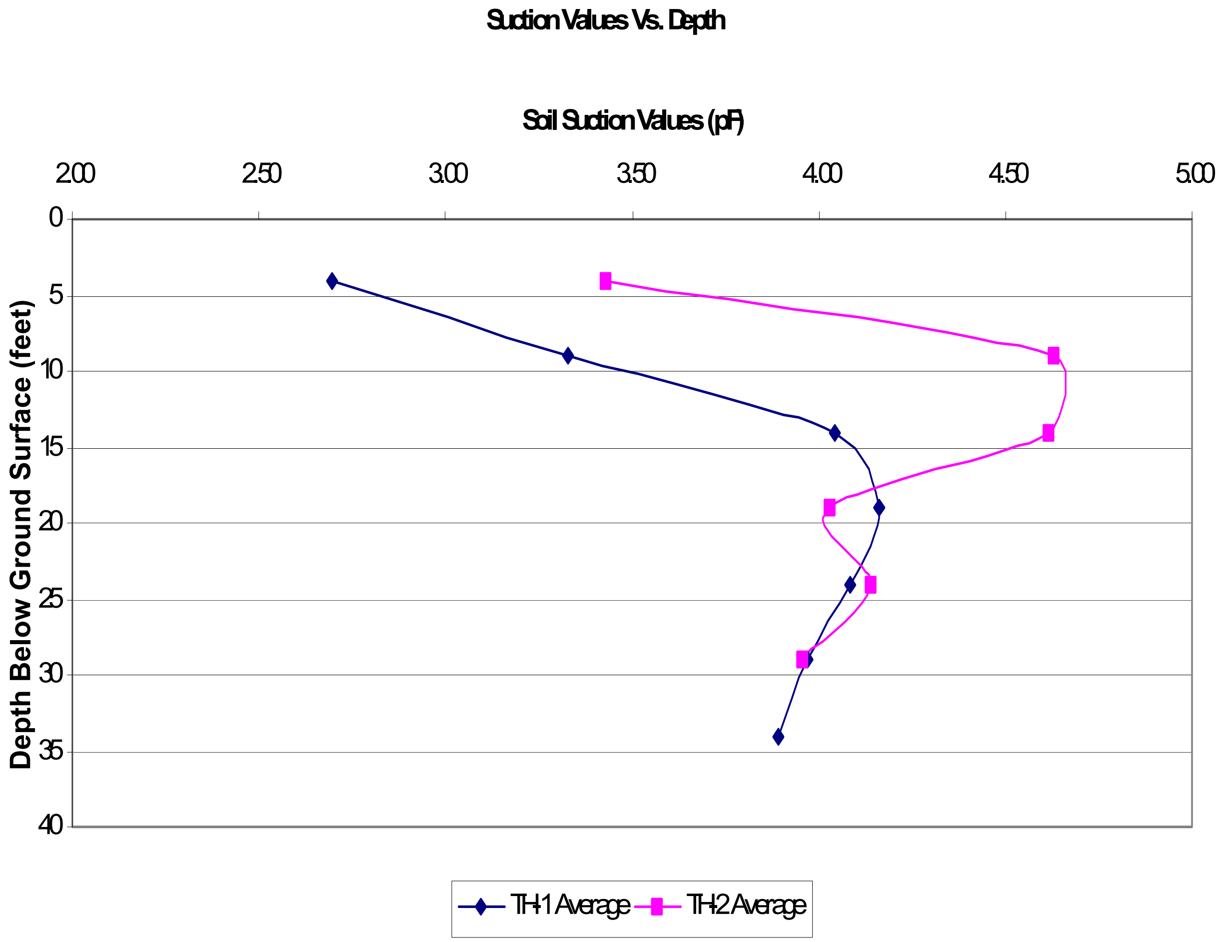

Figure 3. Residential Structure 1 post-construction suction profile.

As discussed above, greater amounts of water are provided through long-term surface irrigation than is permitted to evaporate, so an increase in moisture will likely develop at depth over the anticipated usable life of the structure. Although there was free water in the crawl space and the clay was wetted, the suction profile for the site remains similar to the post-construction wetting profile presented in McOmber and Thompson (2000). The wetting profile for the portions of clay that were not flooded, but are regularly irrigated, suggests development of the predicted long-term wetting profile is occurring. The transient flood event at this site appears to have increased the rate of the subsurface wetting, but not the total depth of wetting.

Commercial Structure 1

Commercial Structure 1 is a concrete and masonry, single-story building with slab-on-grade interior floors. Construction for the building and surrounding landscape began in the middle of 1998 and the building opened in the summer of 1999. Slab-on-grade construction on moisture-conditioned expansive soil and bedrock was selected as the desired floor option. The slab-on-grade floors began to exhibit evidence of heave shortly after completion of construction. The structural distress also was evidenced on interior walls that were attached to floor slabs. Throughout the building, about 1.5 in (3.8 cm) of differential movement is evident between grade beam locations and slabs-on-grade. Survey information indicates that approximately 0.25 to 0.75 in (0.6 to 1.9 cm) of finished-floor heave has occurred since the onset of surveying. A site plan with top-of-bedrock contours is presented on Figure 4.

Storm water drainage is generally flat and occasionally negative on the south and east sides of the building. In addition, irrigated grass is located in a flat area on the southeast side of the building. Ponding storm water in the yard area and on the east side of the building was routinely observed at these locations. An athletic field east of the building was seeded and a considerable amount of irrigation occurred to facilitate grass growth after construction of the building.

The general, subsurface stratigraphy at the site consists of approximately 1 to 6 ft (0.3 to 1.8 m) of very stiff, sandy clay overlying firm to very hard sandstone and claystone bedrock. Ground water was not observed during drilling. Per local practice guidelines, the clay indicated a moderate expansion potential and the claystone indicated a low to high swell potential. The building is founded on drilled piers with a minimum pier length of 24 ft, with a six-inch void below grade beams.

Subsequent investigations determined that a perched ground water table developed during wet periods, and that the heave damage is a result of the upper clay fill and underlying claystone bedrock being subjected to increases in moisture content. Relatively poor surface water drainage existed immediately adjacent to the east and south sides of the building, and storm water was able collect in backfill areas adjacent to the building. Irrigation of the athletic fields, particularly after seeding, also contributed water to the building area. Both of these water sources are located hydrogeologically up-gradient of the building and contribute to the perched ground water condition.

A forensic investigation in 2001 measured expansive fill-soil thickness below interior slab areas from approximately 3 to 5 ft (0.9 to 1.5 m), and flat-lying claystone bedrock was encountered at depths ranging from 5 to 8 ft (1.5 to 2.5 m) below finished grade. The results of suction testing in claystone samples from the site indicate the range in suction values from 3 to 5 pF, and the maximum suction values for the sample interval, decrease with depth. The site is similar to a

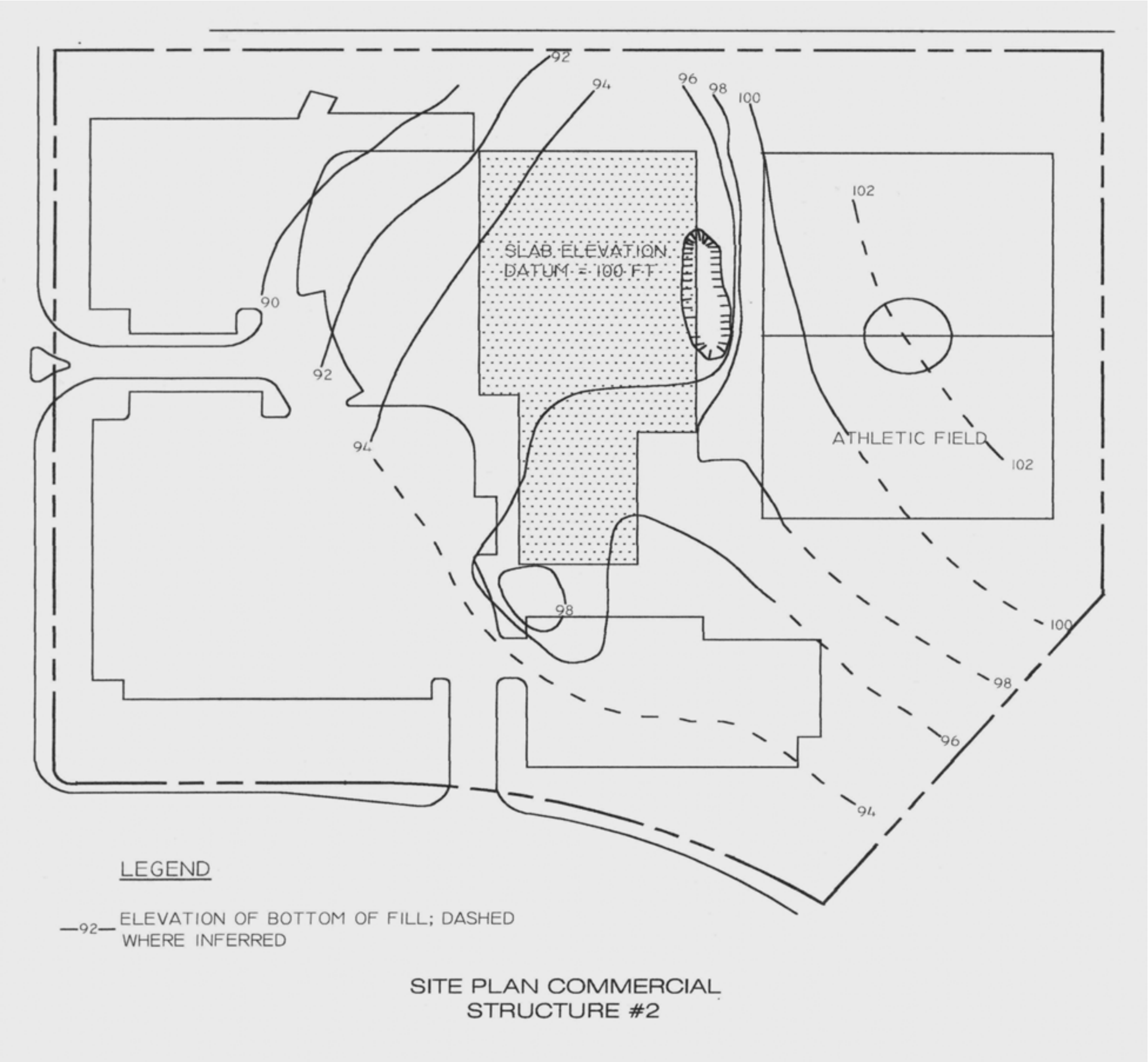

Figure 4. Site Map for Commercial Structure 1.

residential area because it is flanked on the east and southern side by residential subdivisions and the site consists of impervious areas (building, pavements, and slabs) surrounded by irrigated areas.

The claystone did not appear to exhibit a definitive lateral difference in swell potential across the building area and was assumed to present a relatively constant potential for heave throughout the site. For the purpose of heave estimation, the remaining heave potential was estimated using an active zone depth of 20 ft (6.1 m) below grade, based on percent saturation and suction analysis of the soil and bedrock**.** The heave estimates established a potential for 1 to 2.5 in (2.5 to 6.4 cm) of heave remaining within the active zone. The total heave that may occur, based on the actual heave to date and the estimated remaining heave, would be about 4 in (10 cm) or less, which is common for sites with comparable levels of expansion potential.

The data for Commercial Structure 1 indicate a heave potential still remains in the bedrock, while the overlying expansive fill soil was possessed a low swell potential due to very moist conditions. It appears the transient wetting from watering of the athletic fields and surface water drainage created a perched ground water condition that migrated laterally through the fill. The observed heave was likely due to swell of the fill soils and not the underlying bedrock. The swell potential that remains in the claystone bedrock suggests that the wetting events did not significantly infiltrate into the bedrock. As steps are being taken to prevent future transient wetting events, the heave potential that still remains within the depth of wetting in the bedrock will probably be realized over the life of the structure due to loss of evaporation potential and landscape irrigation.

Commercial Structure 2

This building was constructed in 1996 and 1997. The building is roughly circular in shape with a series of separate wings beyond an interior courtyard in the center of the structure. The structure is a one to two-story concrete, masonry, and steel-framed building. The floors are structurally supported and a nominal 3-ft (0.9 m) tall crawlspace exists below most of the building. Outward-facing courtyards with concrete slabs exist within the wings. The interior courtyard is paved with concrete and asphalt, with a thin strip of irrigated grass on the south side. The ground surface surrounding the building is not irrigated for the most part. Irrigated lawn and landscaped areas exist on the east side of the structure around parking lots, and a line of drip-irrigated trees exists north of the building. A minor amount of damage was observed prior to the flooding, including cracking and heaving flatwork and some masonry cracks. A site plan with bedrock contours is presented on Figure 5.

The original geotechnical investigation for the project encountered 7 to 17 ft (2.1 to 5.2 m) of sandy to very sandy clay above claystone/sandstone bedrock. Fill was encountered on the western third of the site. Because the near-surface soils were potentially expansive, piers and structural floors (or structural, fill-supported slab floors) were recommended based on an estimated six inches or more of potential movement if the site soils were wetted after construction. Plans for the project indicated a minimum pier length of 25 ft (7.6 m) and a minimum bedrock embedment of 12 ft (3.7 m).

In spring 1999, flooding of the interior crawl spaces occurred from a broken fire line located on the north side of the building. The pipe was repaired and water was pumped out of the crawl space areas for several days. The amount of water was uncertain, but reportedly several million gallons of water was removed by pumping. Fans and desiccants were then used to dry out the crawlspaces. The extent of associated damage from the flooding was unknown at that time.

Observations of the interior of the building below crawlspace areas were made during the post-event investigation. The maximum heave below grade beams for each crawl space ranged from 0.5 to 5 inches (1.3 to 13 centimeters). Vertical piping located within the crawl space was buckled and offset, which is evidence of soil heave. Shrinkage cracking of surface of soils in the crawl space was observed in most areas, indicating that drying had taken place since the flood event. Two level surveys conducted after the flooding by the structural engineer indicated no significant movement of the structure.

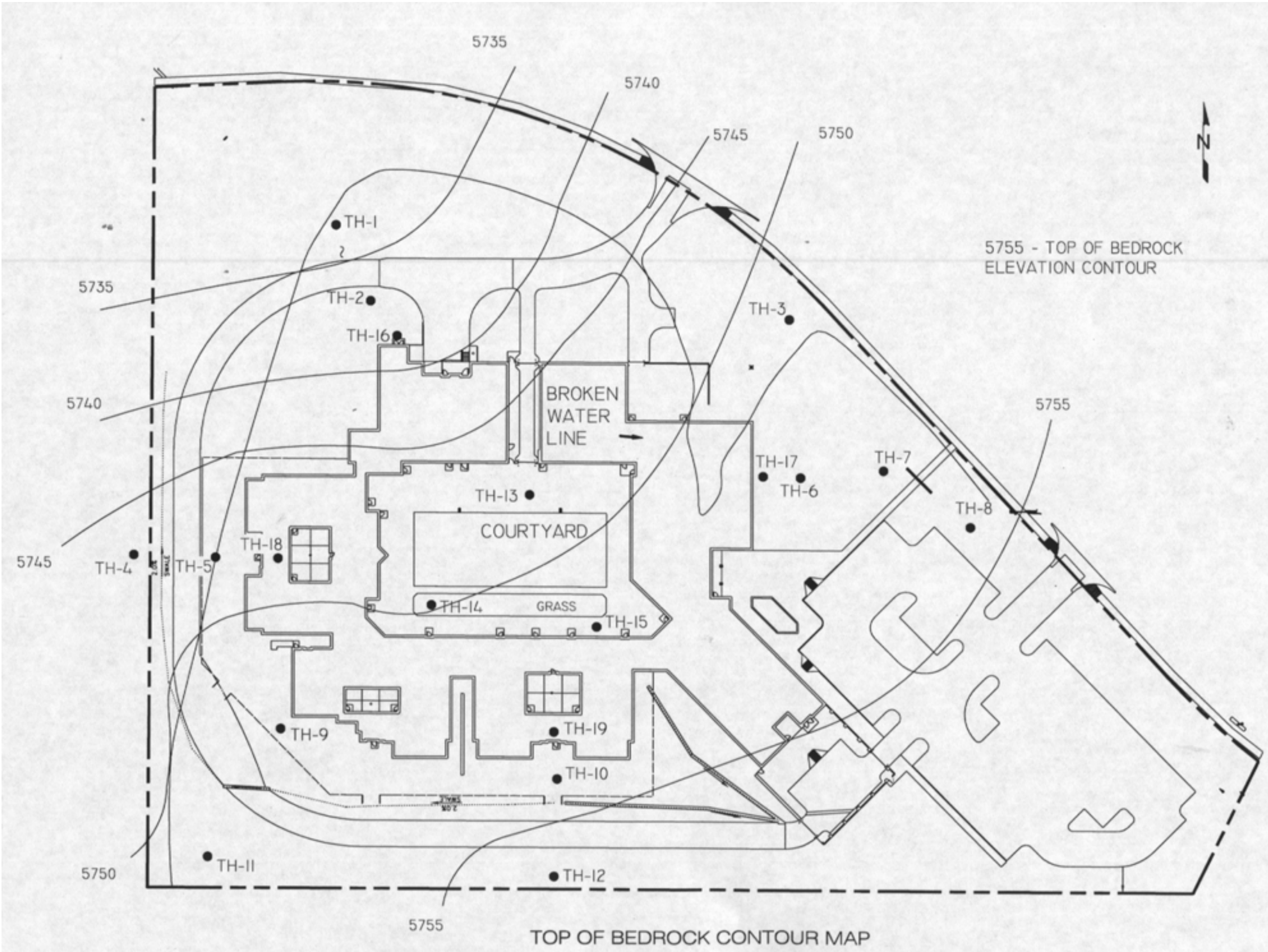

Figure 5. Commercial Structure No. 2 site plan with bedrock contours.

The water from the flood was a pulse of moisture applied to the surface of primarily clayey soils with relatively low permeability. The soil suction results, contained in Figure 6, suggest a depth of wetting of about 20 ft (6.1 m) in the irrigated courtyard areas where flooding did not occur. The moisture content results also suggest a maximum depth of wetting of 18 ft (5.5 m). When the heave estimates for all layers above 30 ft (9.4 m) are summed, 8.4 in (21 cm) of maximum total theoretical heave could occur above a depth of 22 ft (6.7 m), while the interval between 22 and 30 ft (6.7 and 9.4 m) added just 0.2 in (0.5 cm), for a total of 8.6 in (21 cm). If wetting to 18 ft (5.5 m) is assumed, a heave estimate of 7.5 in (19 cm) results.

The data from the forensic study support using the discussed heave estimates determined with wetting depths of approximately 20 ft (6.1 m). Moisture content and suction analysis data do not indicate a deeper depth of wetting assumption from the transient wetting events was necessary for the remediation plans. This conclusion is evident in the review of moisture contents and swell potential of the irrigated courtyard soil and bedrock, where flooding did not occur, compared to similar data for the flooded areas and the non-irrigated exterior soils. These data suggest more complete wetting to a depth of 20 ft for irrigated areas not impacted by the flood. Subsurface moisture conditions in the areas of the flooding were not as moist as the irrigated soils. Therefore, long-term depth of wetting is a function of the irrigation of the site and not the transient wetting that occurs during the flood event.

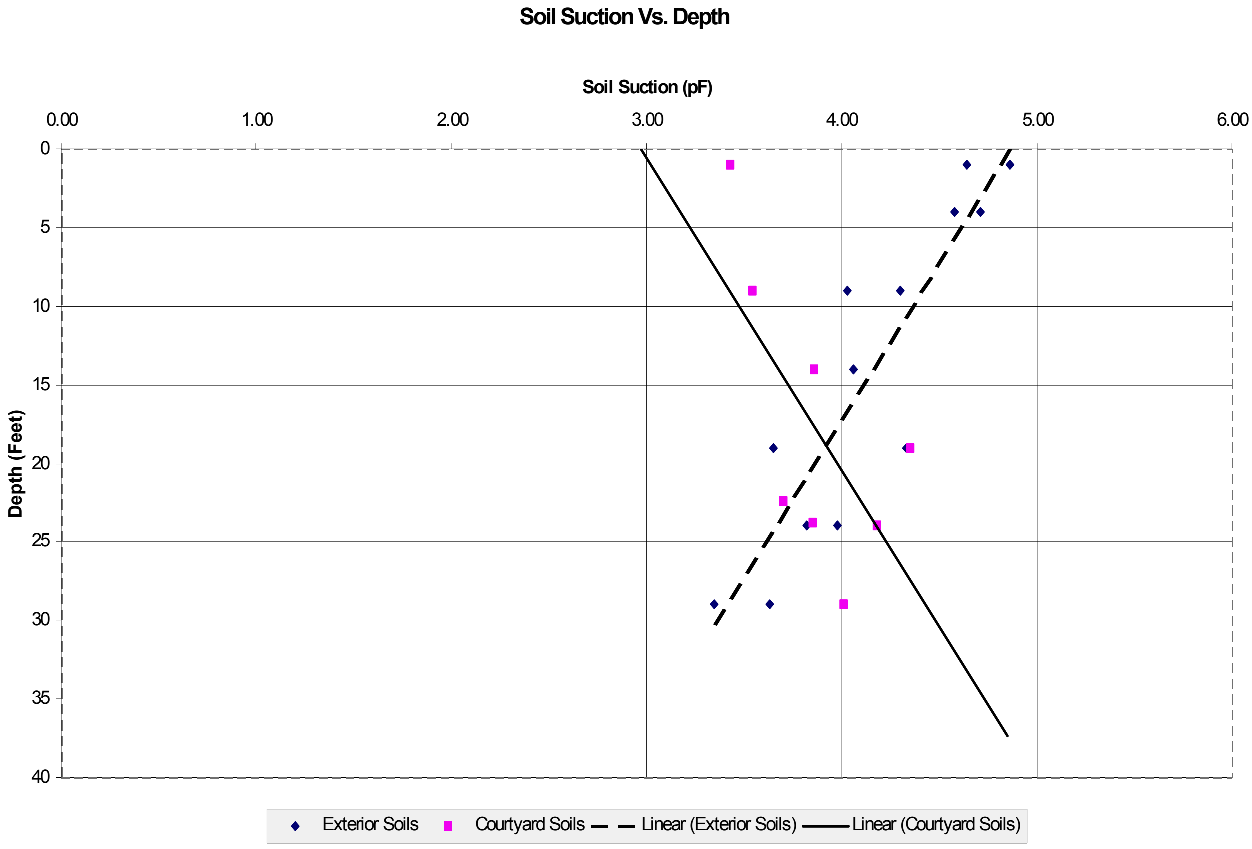

Figure 6. Commercial Structure No. 2 suction relationships.

CONCLUSIONS

The data from the three case studies suggest that the full depth of wetting is not significantly impacted by a transient wetting event when the source of unplanned water is removed on sites with clay soil and claystone bedrock. Long-term irrigation, drainage practices, and reduction of area exposed to evaporation appear to have a greater affect on performance of foundations and slabs. Long-term heave could be effected if the water source is not removed. The reviewed transient wetting events had an impact on the rate of swell of the more permeable, shallow soil. The cumulative swell, which would have likely occurred over several years as a site adjusts to a post-construction equilibrium, appears to be controlled by irrigation and storm water practices rather than a short-term catastrophic event. Soil suction profiling is helpful in defining active zone depth as well as the depth of transient wetting. However, it is very important to evaluate subsurface material composition and variability (site geology) in both the horizontal and vertical directions when evaluating the effects of transient wetting events.

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COLLAPSIBLE SOILS IN COLORADO

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Key Terms: collapsible soils, hydrocompactive soils, low density soils, dispersive soils, settlement, geomorphology

ABSTRACT

Certain soils that form in semi-arid climates in Colorado have a unique property where they compact when they become wetted. This induces ground settlement and potential for damage to structures that are founded on them. In many circumstances, the settlement can be relatively rapid, thence the term collapsible soil. Current research by the Colorado Geological Survey (CGS), which includes compilations of occurrences in the state and analysis of those case histories, has shown that mechanisms of soil collapse vary from mechanical collapse, soils mass loss, and dissolution. Studies of those case histories have also shown that certain types of recent sediment deposits in certain geomorphologic systems can be susceptible to collapse. They are found in geomorphic systems in arid to semi-arid climates where overbank alluvium, windblown sediments (loess), colluvium, and alluvial fan sediments occur, and are generally derived from poorly-indurated, clay- to silt-rich and/or evaporitic sedimentary rocks. They can be quite destructive to foundations, roadways, septic systems, and water diversion and retention structures (canals, irrigation ditches, and dams). Tens of millions of dollars in damage costs have been tallied in Colorado alone by CGS where cost data was available for this study. Actual costs for the state likely run into the $100s of millions. The proper, early identification of these soils is the most important factor in mitigation and design. Since the mid-1980s, geotechnical engineers that practice in these areas in Colorado have begun to recognize the prevalence of these soils and the need to understand the geology, geomorphology, and pedogenetic relationships, instead of narrowly focusing on a few feet of foundation-bearing soils. Subsoil investigations, soil test procedures, and engineering geologic investigations have been adjusted accordingly. While strides have been made, improvements can still be made to improve the state of the geotechnical practice and land use planning to better control damage and distress caused by these soils.

INTRODUCTION

Collapsible soils are unusual in that they have the property to compact, settle, or disperse naturally when moisture is added, generally under very light loads, or just the overburden weight alone. In many circumstances, the settlement can be relatively rapid. Collapsible soils are also referred to as hydrocompactive soils, hydrocompressible soils, low-density soils, meta-stable soils, shrinking or settling soils, and dispersive soils. Research by the Colorado Geological Survey (CGS) (White and Greenman, in prep.), which includes the compilation of 250 case histories, has shown that specific mechanisms of soil collapse differ, depending on site-specific factors including geology, geomorphology, pedogenesis, previous land usage, and climate. Soil collapse mechanisms observed in Colorado include (1) Mechanical collapse, (2) Soil mass loss by dispersion, and (3) Soil mass loss by dissolution.

Studies of case history locations have shown that certain types of recent sediment deposits can be susceptible to collapse. They include (1) windblown deposits of dust, silt, and fine sand called loess; (2) hillside gravity deposits called colluvium; (3) rapid deposition of unsorted water-borne material (mud and debris) in alluvial/debris flow fans or as hillside sheet wash; and (4) stream overbank deposits called alluvium (recent silt and clay laid along the floors of tributary streams and gently sloped mud flats). Collapse-prone soils found in these types of geomorphic systems are generally derived from sediments eroded from clay- and silt-rich and/or evaporitic rocks in an arid to semi-arid climate.

In western Colorado, collapsible soils are a significant soil problem that first impacted early settlers. As early as 1910, a published reference of sinking ground (Paddock and Whipple) was made concerning irrigation and orchard agricultural practices. Failure to recognize these soils and properly design for them has resulted in significant settlement problems for structures built through the energy booms, and recently as population growth and development continues today.

The proper early identification of these soils is the most important factor in mitigation and design. Since the mid-1980s, geotechnical engineers that practice in collapsible soil susceptibility areas in Colorado (Mock and Pawlak, 1983) have begun to recognize the prevalence of these soils and the need to understand the geology, geomorphology, and pedogenetic relationships. Subsoil investigations, soil test procedures, and engineering geologic investigations have been adjusted accordingly. The current state of the practice includes improved foundation designs and improvements in drainage recommendations and irrigation restrictions to control wetting. Improvements can still be made to the states of the engineering practice, landscaping, inspections to verify that recommendations are not ignored, and to appropriate land-use planning so highly susceptible areas can be avoided and the damage and distress caused by these soils can be better controlled.

TYPES OF COLLAPSIBLE SOILS

Three basic types of soil collapse have been recognized in Colorado: (1) mechanical volume reduction (hydrocompaction) of soil structure from the introduction of moisture, (2) soil mass loss from dispersion and piping, and (3) mass loss from dissolution of gypsic and residual soils from evaporitic bedrock. The different types have different engineering properties and generally, but not always, form in different geomorphic systems.

Mechanical collapse, commonly referred to as hydrocompaction, is the predominant form of soil collapse for most granular, low-plasticity soils with higher silt and sand content. These soils mostly form in alluvial fan and hillside colluvial environments, geomorphic conditions that will be further explained in the next section of this paper. Soil grains of a dry, low-density (high void-space ratio) soil become wetted, lose contact strength, and shear against each other to re-orient into a moister, higher-density (lower void-space ratio) configuration, thereby mechanically shifting into a more compact soil (Figure 1). An overwhelming majority of collapsible soil occurrences in Colorado are mechanical hydrocompactive soils.

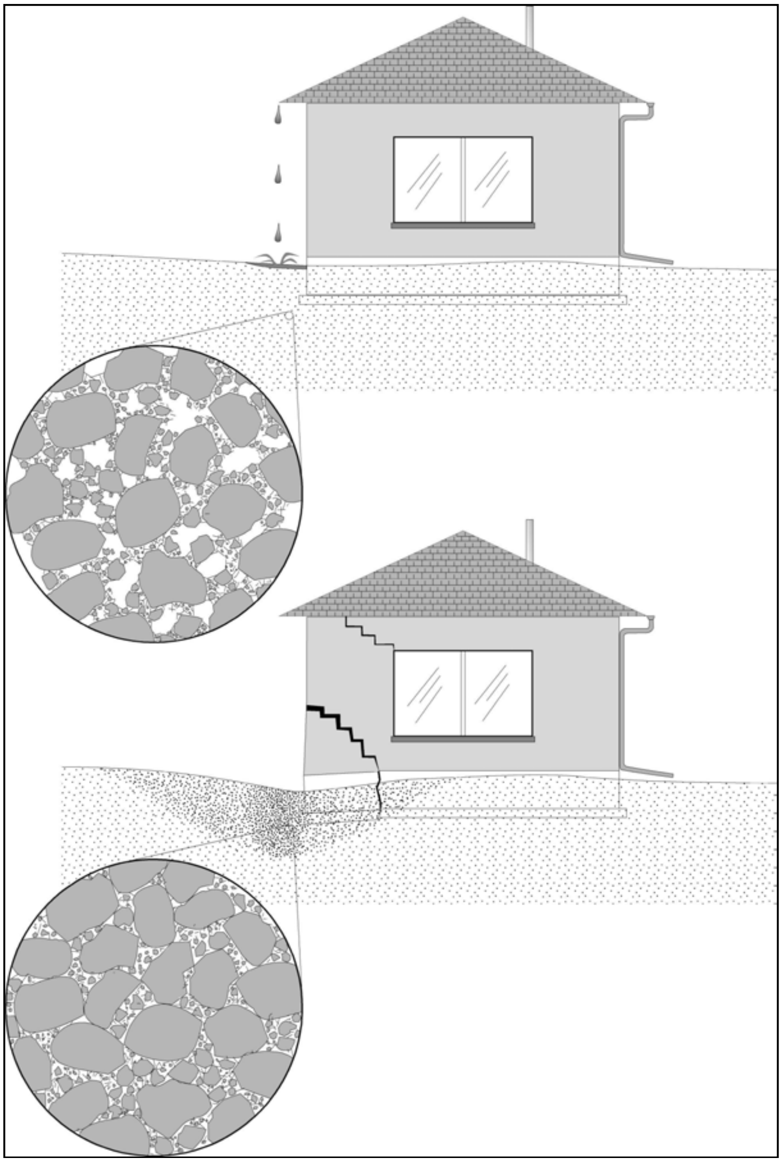

Figure 1. Schematic drawing of mechanical hydrocompaction. Soil skeletal fabric has lost volume from wetting and re-orienting of soil grains, which results in settlement at the surface. From White and Greenman (in prep.), used by permission.

Soil dispersion is another process that manifests itself as collapse. Dispersability is also referred to as colloidal erodability. Dispersive clay soils are those soils where clay and silt particles easily go into suspension when exposed to fresh water. Dispersion susceptibility is a function of weakness of the electrochemical bonds of the clay particles and is governed by the ratios of sodium ions to calcium and magnesium ions. The work by Sherard and others (1976) has shown that dispersion potential increases with higher ratios of sodium ions in the clay. Piping erosion results when passage of water is allowed through the sediments. Those passageways can be the result of holes formed from the decay of plant roots, animal borrows, soil cracks, etc. Generally, soil pipes form when there is a low discharge point for the water flow, such as an arroyo or man-made soil cut slope. These soils are generally unsaturated, have low densities (but not always), and moderate to good shear and bearing strengths while in a dry state. Soil collapse partially occurs by soil mass loss, enlargement of pipes and voids, and subsidence of the bridged material into the void. The resultant landform where soil dispersion and piping occur takes on pseudo-karst morphology (Figure 2).

Figure 2. Sinkhole in clay alluvial soil as a result of soil dispersion and piping near Montrose, Colorado.

The third type of soil collapse is soil mass loss by dissolution. Dissolution of soluble soil constituents results in soil mass loss and settlement of ground surfaces. In Colorado, some of the highest percentages of collapse occur in soils derived from detrital-gypsum sediments eroded from evaporite rocks. Locations of pedogenic, gypsic soils (i.e., gypsum accumulation during soil formation) also present problems in extremely arid locations in west Colorado. Residual, gypsic soils mantling evaporitic bedrock are usually too thin to be of consequence. Subsidence and sinkhole phenomena related to evaporite karst, hazards that also occur in Colorado where evaporite rocks are exposed, are discussed in another paper by the author (White, 2003) in this volume.

The common characteristic of these soils is recent and rapid deposition, i.e., depositional dynamics that result in an inherently unstable internal structure. The generally dry climatic conditions of the area cause these deposits to quickly desiccate (dry out) in their original condition, without the benefit of further re-working or packing of the sediment grains by water. Local ground-water levels generally never rise into these mantles of soil so they never become saturated. Only by human development and land use do local soil-moisture content or ground-water levels rise, through combinations of field irrigation, lawn and landscaping irrigation, capillary action under impervious slabs, leaking or broken water and sewer utilities, and altered drainage.

PROPERTIES OF COLLAPSIBLE SOILS

Collapsible soils are generally dry, low density, low plasticity, silty soils with high void ratios (i.e., space, air gaps, or pores between the soil grains) where the soil-particle binding agents are highly sensitive to water. These pores can sometimes be seen by the naked eye but are most apparent in scanning electron microscopy (SEM) (Figure 3).

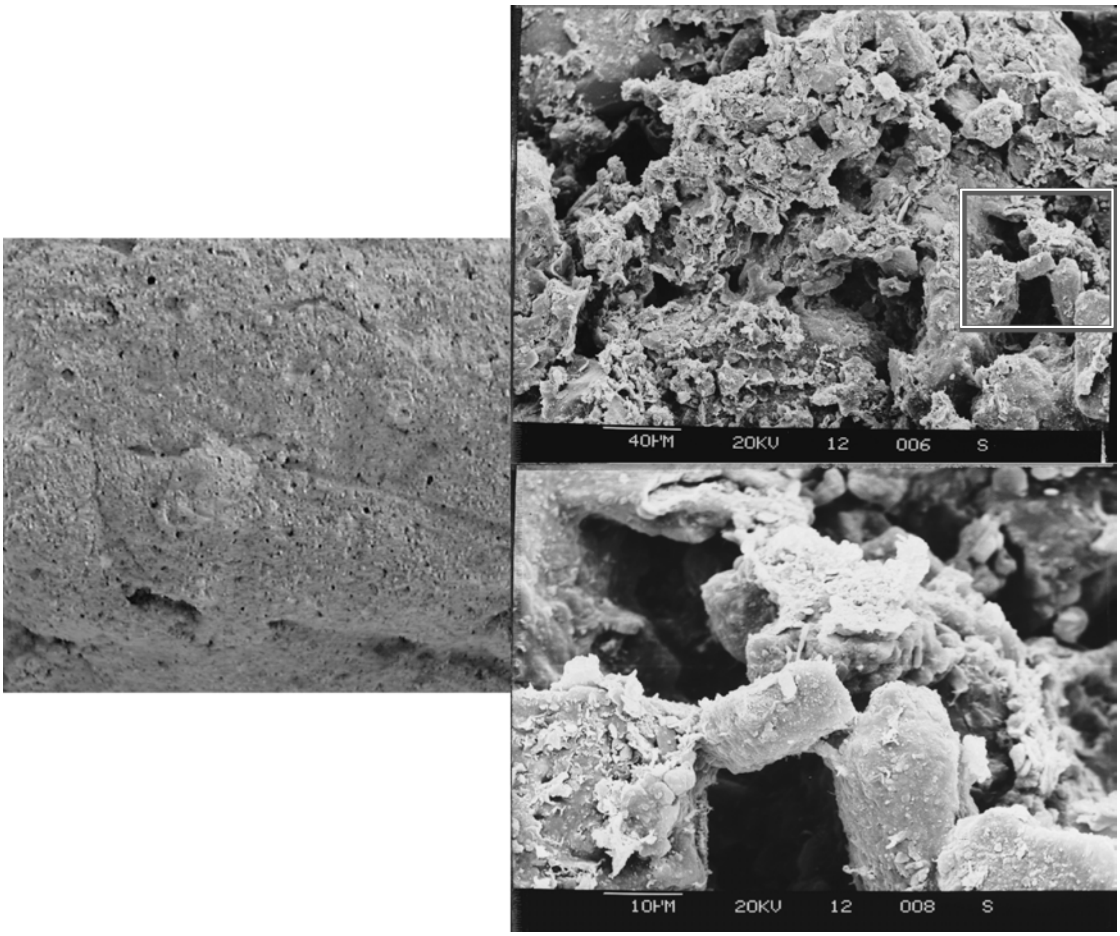

Figure 3. Hand sample on left at normal scale shows visible macro-pores in soil. SEM scan of clayey silt (CL-ML) on right shows abundant micro-pores and high void-space ratio. Close-up scan (scale bar in microns) on bottom right was taken from boxed area above. Note clay bridges supporting silt grains. SEM images courtesy of R. Luehring.

The soil-bonding agents can be quite strong, and may possess high bearing capacities able to support heavy structures, but weaken quickly in the presence of water. Beckwith and Hansen (1989) used an appropriate term, that the sand and silt grains are “tack-welded” in their loose honeycombed state by clay particles, chemical precipitation, and/or soil suction pressures (capillary tension) of the thin water film at grain contacts called the meniscus. When water is introduced the soil suction pressures diminish, and/or the binding agents break, soften, or dissolve such that the soil fabric’s skeletal structure quickly weakens and fails. The soil grains shear against each other and re-orient in tighter, denser, configurations. This re-configuration causes a net volume decrease in the soil mass that, in turn, results in settlement of the ground surface (see Figure 1). This condition can occur just by the weight of the soil itself, called the overburden pressure, or the weight of a structure, such as a home foundation, concrete or pavement slab, or dam abutment. The above process meets the criteria of Barden et al. (1973) for collapse of soil structure: (1) an open, potentially unstable structure; (2) a high enough applied stress (load or weight) component to develop a metastable condition; and (3) a suitably strong soil bonding agent to hold and stabilize the soil grain contacts in their original, precarious meta-stable orientation.

Collapsibility is commonly determined by one-dimensional swell-consolidation tests (modified from ASTM D-2435 and D-4546 soil-testing methods). In this test an "undisturbed" soil sample is collected, generally by driving a metal cylindrical sampler into the soil mass. The samples are hydraulically jacked into a confining ring, trimmed, and inserted into a test chamber with porous inserts at top and bottom. The sample is then loaded with weights to a specific pressure, commonly 1,000 psf (48 kPa). The soil is then saturated and allowed to drain, and the percent collapse or swell is recorded at that constant pressure. The soil is then further incrementally loaded to determine the compression curve. Severity criteria for soil collapse, based on a percentage decrease of a sample height before and after saturation, was developed by Jennings and Knight (1975).

Mock and Pawlak (1983) locally established the following collapse potential based on their own experience in west-central Colorado and, in part, based on Jennings and Knight (1975), but at a reduced load of 1000 psf (48 kPa) when the test sample is saturated; this load criteria has become the standard practice in central and Front Range, Colorado.

Percent Collapse	**Hazard**
0-1%	No Problem
1-3%	Low
3-5%	Moderate
>5%	High

Swell-consolidation testing of soil samples in the laboratory, or field soil-saturation plate load tests, will give the most accurate determination of collapse potential and related hazard potential. There are a number of other analytical methods, however, that have been developed by researchers to identify collapsible soil. These are based on comparisons of measurable soil material properties such as dry density, liquid limit, moisture content, degree of saturation, porosity, void ratio, specific gravity, soil gradations, etc. Published papers by Roullier and

Stilley (1993) and Leuhring (1988) present good synopses of these various methods. This body of work has resulted in certain empirical generalizations that have been verified by the examination of occurrences in Colorado: common collapse-prone soil properties are low densities, low native moisture content, and low plasticity (low liquid limits). Soil densities can be as low as 70 pcf (1.12 g/cm^3) in some of the low-plasticity clayey to sandy silt soils, but generally range in the low 80's (1.28 g/cm^3) to middle 90's (1.44 g/cm^3). These soils are very dry in their native state with moisture contents in the 2 to 6 percent range. The plasticity index (PI) generally runs in the single digits so the soils commonly fall into CL-ML, ML, and CL categories in the Unified Soil Classification System, becoming SM-SC and GM-GC where they contain a sizable sand and gravel component, which is common in alluvial fans and hillside deposits.

There are some precautions about over-reliance on swell-consolidation testing. Artificial soil densification by the driving of the thin-walled sampling tube into very low-density soils may skew consolidation results. Also, high clay content soils may have a swell component under very light loads, although the saturation of a test sample at 1,000 psf (48 kPa) will indicate only collapse. In fact, this property has resulted in regional differences in swell-consolidation testing practices that will be discussed later. High gypsum content soils may not reveal immediate collapse during the normal time interval for post-wetting in the swell-consolidation test because of the time lag for dissolution.

GEOMORPHIC PROVINCES WHERE COLLAPSIBLE SOILS OCCUR

Certain conditions are needed for the formation of surficial deposits (soils) that have the ability to collapse, settle, or compact. As was explained above, these soils are generally very dry (i.e. low natural moisture content) and exhibit low density or a honeycomb soil fabric with high void space ratios in their natural state. As a general rule, collapsible soil formation occurrences in Colorado are in arid to semi-arid environments where poorly indurated, clay- and silt-rich sedimentary rocks are present. Except for windblown loess deposition, there needs to be a suitable higher-elevation area above depositional areas to provide source materials for sediments. These sediments are eroded from the source areas and then deposited by wind, rain, gravity, and flowing water to valley walls and floors, basins, swales, drainageways, and other low-lying areas (or gently-sloped areas where loess blankets can be deposited) where sediments can accumulate. To achieve the honeycomb structure of the soil, the sediments, if not windblown, need to be deposited quickly and vigorously, and then quickly desiccated and become thoroughly dried before subsequent deposition takes place. Textbooks concerning the urban geomorphology of dry lands and desert geomorphology make passing comments on the formation of meta-stable soils in these types of environments (Cooke et al., 1982; Cooke et al., 1993).

Arid to semi-arid climates generally produce high sediment yields and subsequently high erosion rates. Researchers such as Beckwith and Hansen (1989), among others, have pointed to the work by Langbein and Schumm (1958) that describes much higher sediment yield in semi-arid lands where annual precipitation is depressed, compared to more moderate temperate areas. The graph in Figure 4 from Langbein and Schumm compares sediment yield with annual precipitation. Semi-arid areas have less vegetation but are still exposed to episodes of intense thunderstorms, which are able to generate sufficient runoff to transport large amounts of sediment. This is typical of Colorado where debris flow hazards are pronounced. Langbein and Schumm's graph shows peak sediment yield at an annual rainfall of about 12 inches, typical for much of western Colorado in areas that are not in higher elevation mountainous zones.

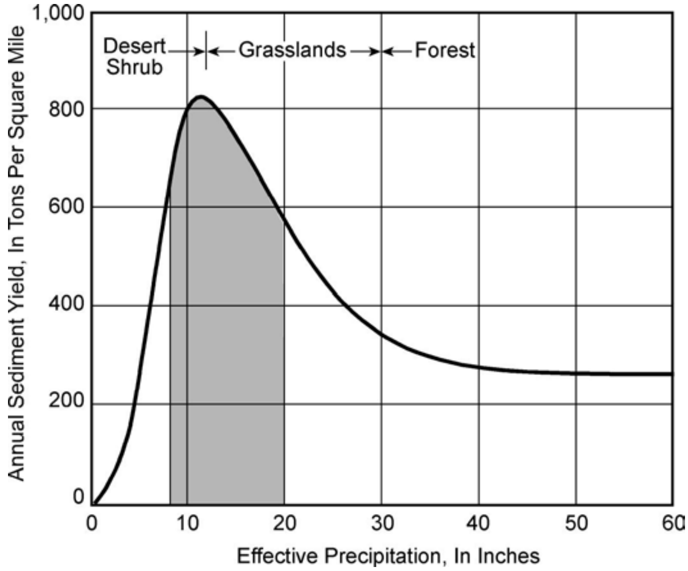

Figure 4. Sediment yield chart modified from Langbein and Schumm (1958). Shaded area of high sediment yield corresponds with precipitation values for most areas of Colorado outside of the high mountain regions.

As can be seen in the figure, sediment yield drops off radically in very arid areas. Intense thunderstorms rarely occur in very dry areas, so the sediment yield is minimal, even with lack of vegetation. Conversely, sediment yields (erosion) decreases gradually with increasing rainfall as the climate becomes more moderate and thicker vegetation covers and protects the ground surface. In many provinces of Colorado, gypsiferous or high-salt or sulfate shale bedrock is exposed where only the hardiest of plants can survive (Figure 5). Such sparsely covered terrain, much of which could be called badlands, is even more prone to erosion and is subject to high sediment yields.


Figure 5. Areas nearly barren of vegetation in Colorado. Photo on left shows badlands underlain by Mancos Shale near Montrose. Photo on right shows exposures of Eagle Valley Evaporite bedrock in Roaring Fork Valley near Carbondale. Photos from White and Greenman (in prep.) used by permission.

Much of Colorado has the necessary requirements for the generation of collapse-prone soils: Wide expanses of clay- and silt-rich, poorly-indurated rock formations; a semi-arid environment where intense thunderstorm cells can occur; and steep slopes, where sediments are quickly eroded and quickly deposited below. High sediment yields have created thick accumulations and aggradations of more-recent Holocene deposits. These young sediments have been deposited with the honeycombed, meta-stable structure referred to earlier and, because of the arid state of the climate, these soils never naturally become saturated or exposed to ground water. Pedogenic, biogenic, and man-induced factors also play a role in collapse-susceptibility of these types of soils.

The major discussion in this section addresses the geomorphological processes, landforms, and the soil development where collapsible soils can form in Colorado. Earlier, defining work on western US geomorphology where collapsible soils sediments are formed includes Lofgren (1960, 1969), Bull (1964), Beckwith and Hansen (1989), and Rollins et al. (1992).

The methodology for the CGS statewide collapsible study (White and Greenman, in prep.) included examination of the local geology, geomorphic terrains, and soil deposits at locations where collapsible soils were known to occur. This compilation verified that certain types of geologically recent surficial deposits are prone to soil collapse. Attention increasingly focused on Holocene and Late Pleistocene deposits, generally no later than Late Wisconsin or Pinedale glacial ages, generally less than 15,000 BP. In examination and compilation of natural collapse and/or subsidence case histories, we find the following geomorphologic systems closely relate to occurrences of various forms of collapsible soils:

1. Alluvial fan/Debris fans
2. Colluvial slope deposition
3. Holocene fluvial flood plains
4. Eolian deposits (loess);
5. Gypsiferous soils and evaporite bedrock
6. Near surface weathering and alteration of gypsum-bearing Mancos Shale.

The block diagram illustration shown in Figure 6 reflects many of the landform types and surficial deposits in Colorado where collapse soils may exist.

PUBLISHED RECORD OF COLLAPSIBLE SOILS IN COLORADO

The published record of collapsible soils in Colorado, for both classic hydrocompactive soils and piping and dispersive soils, extends to the early 1900s. Most of the work coincides with the post-WWII construction boom, the energy booms of the 1970s and 1980s, and recent 1990s booms in residential construction. Figure 7 shows the geographic locations of previously published references in Colorado. The points are numbered and indexed to a citation table. A synopsis of the content of these published reports cited will be available in White and Greenman (in prep.), which will soon be published by CGS.

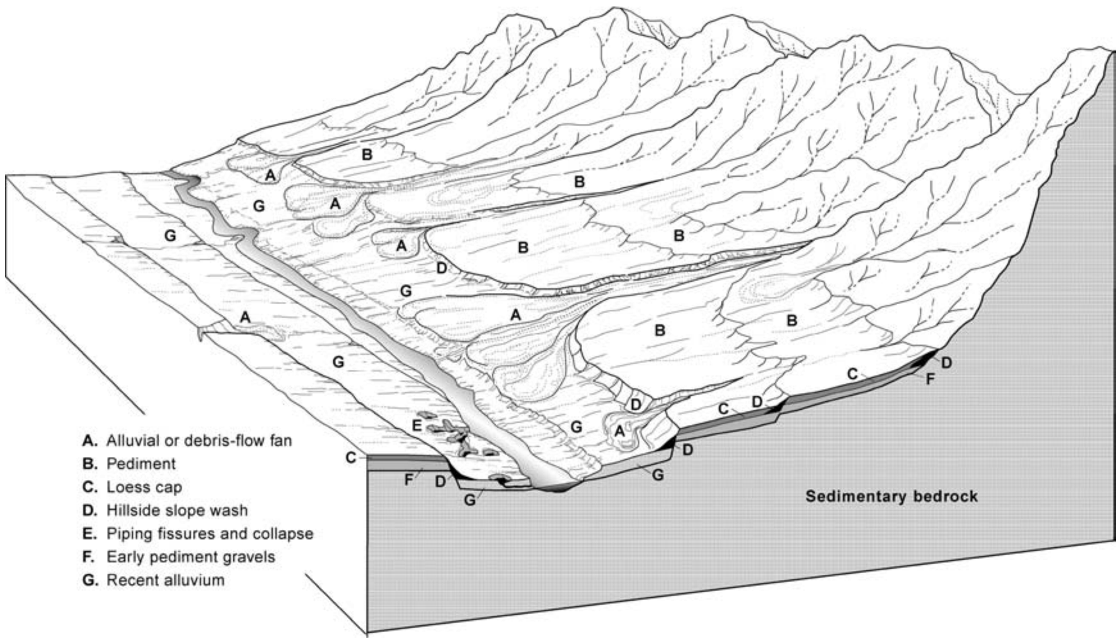

Figure 6. Block diagram of typical Colorado landforms where collapse-prone soil may develop, modified from Beckwith and Hansen (1989). From White and Greenman (in prep.), used by permission.

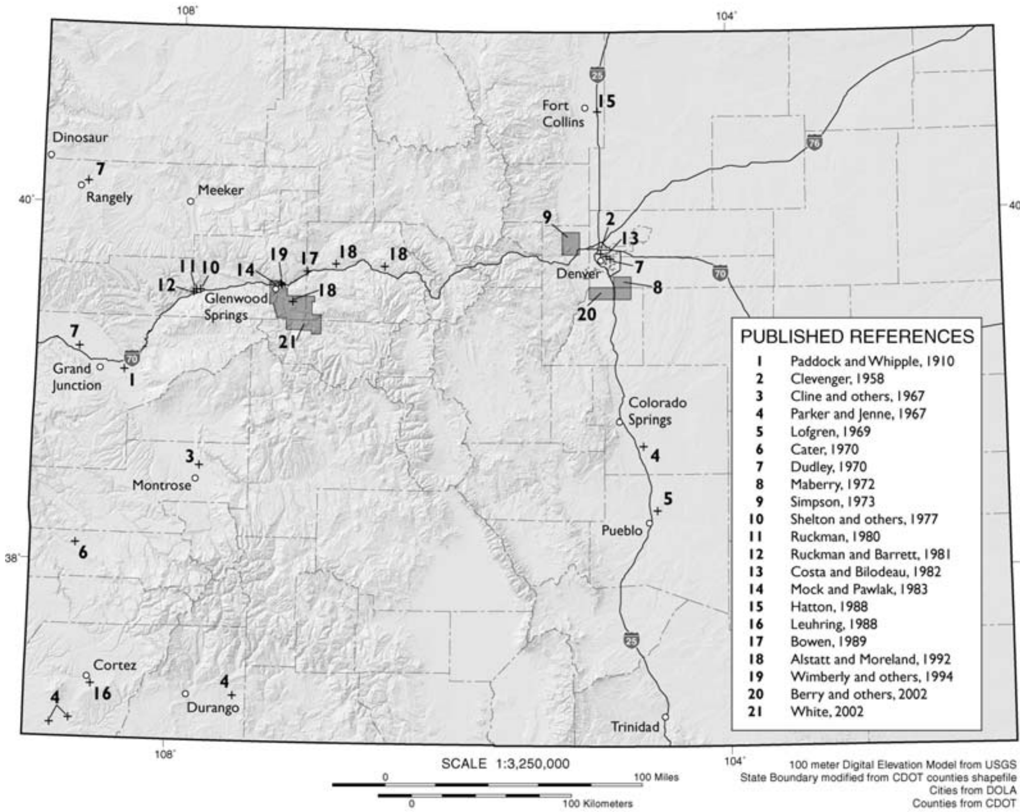

Figure 7. Map showing locations of published references of collapsible soil in Colorado. From White and Greenman (in prep.).

Of considerable interest is the first-known published account of collapsible soils in Colorado; this is an interesting early text on farming practices in the semi-arid western states, specifically fruit orchards and irrigation of dry soils that were previously never exposed to saturation or high ground water. While not understood at the time, early fruit growers and agriculturists began to recognize the hazards of "sinking ground," and early horticulturists made one of the first references to collapsible soil in 1910. Wendell Paddock (Professor of Horticulture at the Colorado Agricultural College and Experiment Station in Fort Collins, the precursor to Colorado State University) and Orville Whipple (Field Horticulturist in Grand Junction, Colorado) discussed the hazards of breaking out land for irrigated fruit orchards, specifically in the Grand Valley area near Grand Junction. They discussed at length the phenomenon of "sinking land." The following is excerpted from their book (Paddock and Whipple, 1910):

> *Sinking Land*
>
> *Land that settles when water is applied is known as sinking land. Some of the highest-priced peach orchards are located on such areas. To all outward appearances this land does not differ from that found in many other places. No hint as to this peculiar characteristic is gained from the general looks of it; but when irrigation is attempted, irregular patches, here and there, settle four or more feet, and in some cases cracks occur that may extend into the ground to a depth of fifteen feet. Such an occurrence is surely alarming, to say the least, to the uninitiated.*
>
> *In one locality, where there is a small tract of such land, the owner attempted to establish an orchard, and planted the trees before the land had been irrigated. At the first application of water, spots of land here and there began to sink, and deep cracks were formed. Of course it was impossible to save the trees with the land in such condition, and the owner was obliged to give up. This type of land may usually be "settled' in one season if water is persistently applied. It often requires more time, however, and as the 'settling' is very uneven, much leveling is required in order to fit the land for cultivation. The tendency to settle appears to be due to the porous condition of the subsoil.*

They go on to state that, *"The tendency to settle appears to be due to the porous conditions of the subsoil."* Such visible porosity (i.e., Figure 3) and soil properties are diametrically opposite from the swelling problems that are found in "fat" plastic clay soils along the Front Range.

The description of the subsidence, downwarping, ground cracks, and porous nature of the soils are good approximations of what is commonly observed with classic hydrocompactive soils. Paddock and Whipple (1910) also make one of the first recommendations for mitigation of collapsible soil hazards, prewetting. Simply stated, when breaking out new land for fruit orchards, the fields should be flood irrigated for a suitable time to induce soil collapse. This is to be done before final grading of the orchard field, irrigation channels excavation, and planting the fruit tree seedlings.

COLORADO PROVINCES SUSCEPTIBLE TO COLLAPSIBLE SOIL OCCURRANCES

The spatial distribution of case histories of collapsible soil shown in Figure 8 is revealing when shown on a map of Colorado, draped on a shaded relief background and superimposed with precipitation data. The case history data is, of course, skewed with more incidents in higher-growth areas in the state, but several generalizations can be made.

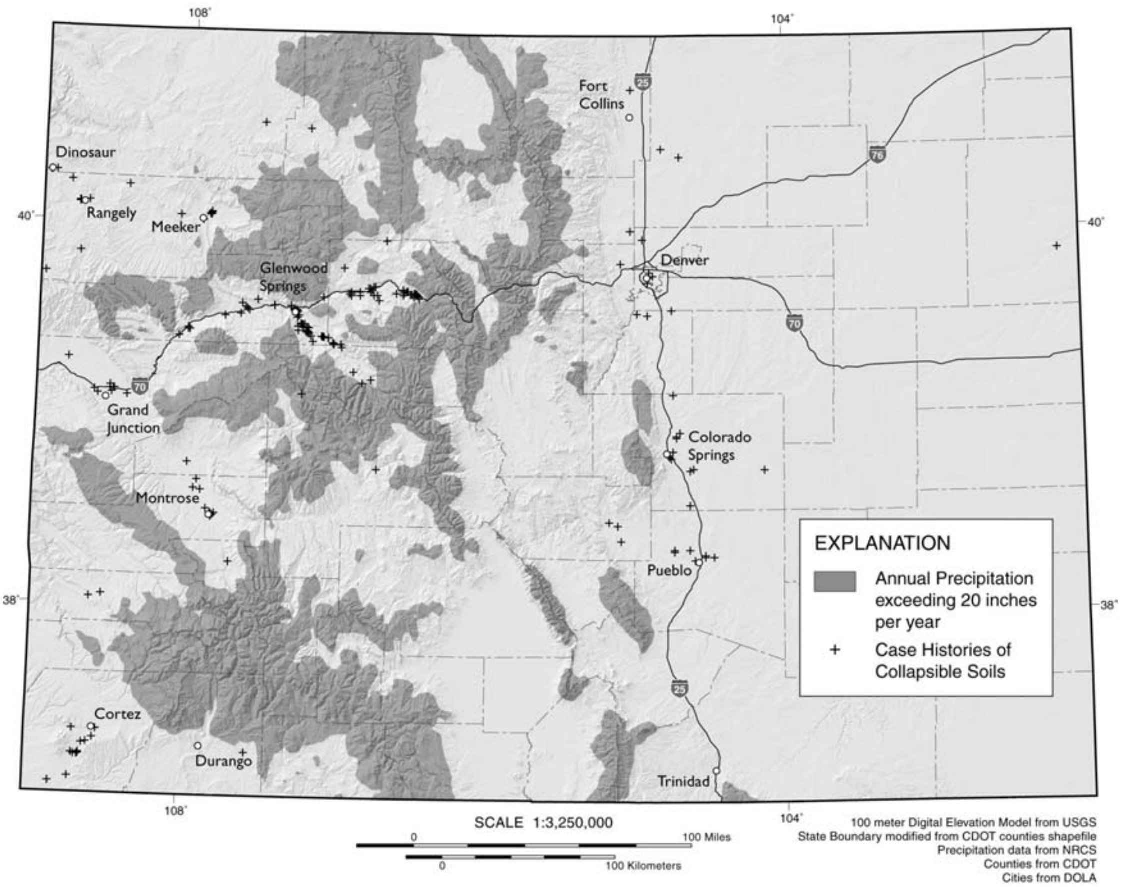

Figure 8. Map showing locations of compiled case histories of collapsible soil occurrences in Colorado. From White and Greenman (in prep.), used by permission. Nearly all locations are in areas of less than 20 inches (508 mm) of annual precipitation.

- Statewide, collapsible soils generally do not occur where annual precipitation rates exceed 20 inches (508 mm) per year, with most occurrences lying in the 12 to 16-in (305 to 406 mm) per year range. They also do not generally occur in crystalline and volcanic rock terrains of Colorado.

- In west-central terrains of Colorado, soils with collapse potential tend to form in aprons and wedges of colluvial sheetwash, hillside colluvial wedges, and coalesced alluvial fans that mantle valley walls and floors, in semi-arid climates where packages of poorly indurated clay- and silt-rich sedimentary rocks, or evaporitic bedrock are exposed. Most of these valleys and major tributaries contain treads of middle to late Pleistocene glacial outwash on the valley floor and erosional pediments that have beveled the hills above. Blankets of loess and additional surficial soils from the bedrock hills above have, in turn, mantled these terrace and pediments surfaces. Figure 9 is a block diagram based on a digital elevation model of a portion of the Roaring Fork River valley in west-central Colorado, reproduced from White (2002). It helps illustrate the topography and geomorphology where formation of surficial deposits can be susceptible to soil collapse. Recent soils that mantle the valley sides of the Colorado, Eagle, Roaring Fork, White, and Yampa River valleys, and other major tributaries that pass through the same sedimentary and evaporite formations should be considered as being potentially susceptible to soil collapse. Similar circumstances occur along the Dolores River valley in southwest Colorado where it passes though the Salt Anticlines area. For more information on collapsible soil hazards in the Roaring Fork River valley, see the paper by Lovekin and Higgins (2003) in this volume.

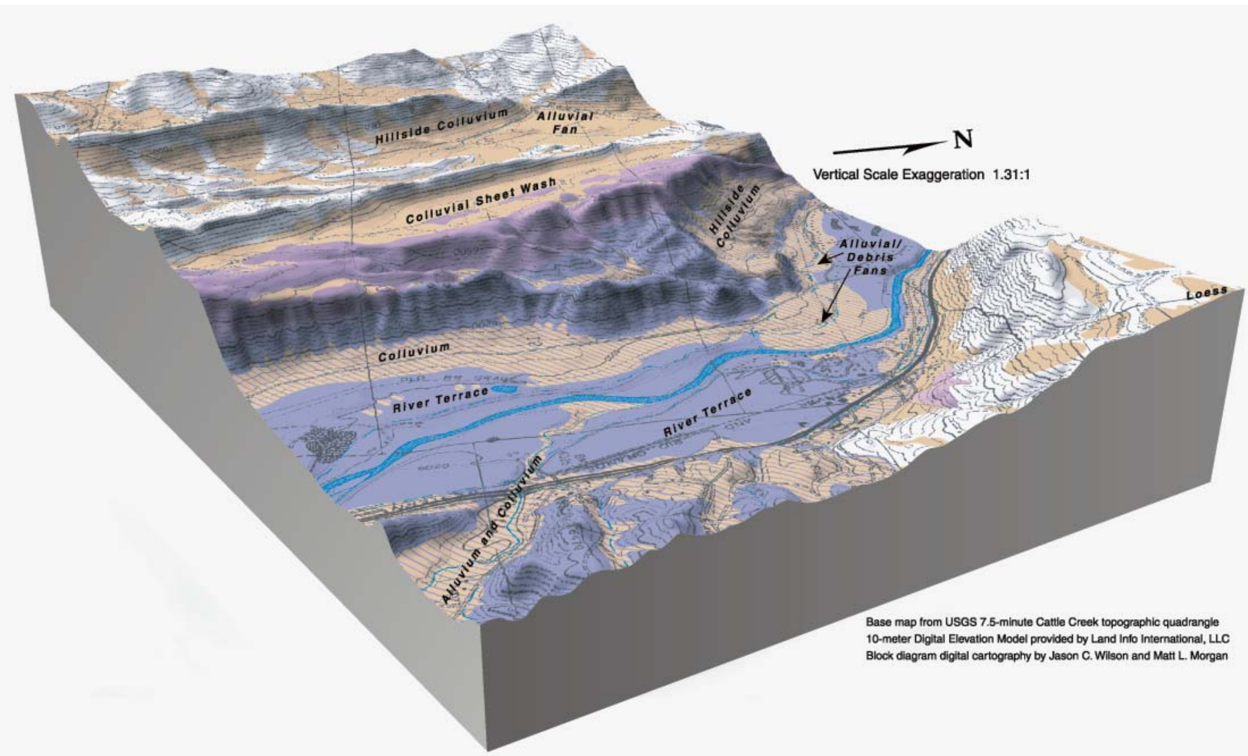

Figure 9. DEM model of segment of the Roaring Fork Valley between Glenwood Springs and Carbondale that illustrates the geomorphology where collapsible soils can form. Annotated tan-colored (light) areas are surficial soils susceptible to collapse. Blue and purple shadings (dark) are mapped areas of Eagle Valley Evaporite bedrock. From White (2002), used by permission.

- In the western Colorado plateau region, large broad valleys form where the thick Mancos Shale formation is the surficial bedrock unit. Prominent lowlands occur at Mancos Valley and Montezuma Valley near Cortez; the Grand and Uncompahgre Valleys that extend from the Utah border through Grand Junction and on to Montrose; and the broad White River Valley near Rangely. The surficial soils are formed from large broad mudflow alluvial fans, mud flats, and aggraded fine-grained alluvium eroded from shale badlands, now commonly gullied by tributary streams and arroyos. They can have high percentages of gypsum and alkali salts. Such very fine-grained soils in generally arid climates are prone to hydrocompaction and hydroconsolidation, dispersion, piping, and creation of voids and surface subsidence features that are collectively called pseudo-karst morphology (Figures 2 and 10).

Figure 10. Soil dispersion and dissolution has created a pseudo-karst morphology near Loutsenhizer Arroyo near Montrose. Photo from White and Greenman, (in prep.) used by permission.

- Along the Front Range, while overshadowed by the well-publicized problems with swelling clay soils and heaving claystones, collapsible soils also occur. Work by White and Greenman (in prep.) has shown that most of these soils are a result of eolian deposition, reworked eolian sediments, and colluvial slopewash. Varying modes of sediment transportation and deposition result in changes in soil properties. At times, complex interlayering of swelling and hydrocompactive soils (i.e., thin loess sheets within clayey colluvium at the base of slopes, or interfingering alluvium and colluvium within shallow swales) can occur, which makes subsurface investigations and forensic investigations at distressed structures difficult. Recent work by Berry et al. (2002) in the Highlands Ranch area of the southern Denver metropolitan region illustrates the proximity of highly collapsible and highly expansive soils.

DESIGN ASPECTS IN COLLAPSIBLE SOIL TERRAIN

Where the geologic and geomorphic conditions suggest collapsible soil, a certain level of geotechnical investigation is warranted. The scope of this paper does not allow an in-depth discussion of design aspects, but the following are some observations that have been made by analyzing case histories during the research done for the statewide program and noting common mistakes or errors in judgement.

One of the most significant criteria in subsurface investigation is to determine the thickness of the collapsible soil. Too many investigations evaluate only the subsurface conditions for an already-assumed shallow foundation, with termination of boring depth, or shallow test pit, approximating where the influence of the stress-distribution envelop or bulb becomes insignificant at the assumed bearing load. With collapsible soil, *which can easily compact under the weight of its own overburden when wetted*, the total thickness of this type of soil is important and should be verified. Even low collapse potential, 1 to 2 percent, can create problems if the collapse soil column is thick enough and a wetting or saturation front ultimately extends deeply into it. A 15-ft (4.5 m) column of 2-percent collapse soil, if fully saturated, would result in 2.5 inches (90 mm) of settlement, and if that settlement is uneven across the structure, it could cause significant damage if mitigative measures were not taken.

Certain collapsible soils can be very coarse-grained, with upwards of 75 percent gravel and cobble-sized rocks. Such circumstances occur in hillside colluvial wedges and alluvial fans. Close examination of these gravelly soils reveals that the disseminated gravel, deposited as a debris or mudflow, is supported in a clayey silt matrix, which can be collapsible. It becomes very difficult to retrieve an "undisturbed" sample using thin walled tubes in these types of soils for typical swell-consolidation tests done to determine collapse potential. For large or critical facilities, plate load tests conducted by saturating the subsurface soils with ponded water and infiltration wells will yield a more accurate assessment of future settlement.

Collapsible soils are different depending on the formational source. More-granular soils derived from silt-rich valley walls can have very high rates of immediate mechanical collapse upon wetting, resulting in a flatter, post-saturation consolidation curve upon additional loading. Other, more cohesive clay-rich soils, such as the fine mud alluvium derived from the Mancos Shale, may have little immediate collapse upon wetting, but very steep consolidation curves upon additional loading. These clay soil-types may actually be slightly expansive at very low loads upon initial wetting, where the clay particles expand upon hydration but the soil binding agents have not yet broken and sheared. Further loading will cause mechanical compaction and long-term consolidation as the collapsed clay soil then begins to expel the pore water. For this reason, there are regional differences in swell consolidation testing methodology in Colorado.

The standard of practice for swell-consolidation testing for swelling-soil regions along the Front Range and the west-central valleys prone to hydrocompactive soil is to load the soil sample to 1,000 psf (48 kPa), after which the sample is then wetted. In western Colorado, where the soils are more clayey yet still settle under load when wetted, local geotechnical firms load their soil sample to only 100 psf (4.8 kPa) or 250 psf (12 kPa) to examine the soil behavior and degree of expansiveness at very light loads, prior to measuring the steep consolidation curve upon additional loading. In the Montrose area, there are circumstances where the subsoils have been wetted and the sidewalk has heaved while the adjacent building on a shallow foundation has significantly settled. Similar phenomena occur along the Front Range with clayey colluvium and reworked clayey loess. While counter-intuitive, this behavior results from slight swells upon wetting with no or very light loads (i.e. sidewalk slabs), while the bearing pressures of the heavier foundation load has induced soil collapse upon initial wetting, and further settlement from consolidation of the now-wet, clayey soil.

Drainage and water-management design criteria are extremely important. Too often it is the unintentional or accidental flooding and saturation of subsoils that causes damage to structures. Utilities such as water mains and sewer lines should be carefully designed and reinforced where they pass through collapse-prone soils. Roof gutter downspouts should discharge away from foundation walls. The same holds true for irrigation; sprinkler heads should not be placed near foundation walls or water allowed to splash against them. Positive grades away from foundations must be established and maintained.

Even today, with the knowledge of collapsible soils and the settlement damage that they can cause, inadequate foundation type and poor drainage design can result in significant damage. For example, a large complex of townhomes in Glenwood Springs that was just completed last summer (2002) is currently experiencing significant settlement damage (Figure 11). Even though the underlying soils were well known to be potentially hydrocompactive, these townhomes were built on standard spread footings (R. Grance, Glenwood Springs Building official, pers. comm.) with very poor positive drainage on the backside of the units and, astoundingly, no rain gutters and downspouts for the portion of the roofs that slope that way. As rain collects and flows to the roof edge, it falls as curtains of water, pooling and saturating the soils along the back of the units. Significant ground settlement and structural distress has now begun (and will continue without immediate landscaping and drainage improvements) that will require remedial foundation design and repair.

The scope of this paper will not allow an in-depth discussion of many different types of mitigation methods for collapsible soils. In summary, they maybe grouped broadly into: 1) ground modifications that reduce the collapse potential of the soil (such as prewetting, excavation and compaction, dynamic compaction, compaction grouting, etc); 2) structural reinforcement of shallow-type foundations; and 3) deeper foundations that transfer building loads through the collapsible soil to a competent soil or rock layer below. Additional information on mitigative measures for collapsible soils can be found in Clemence and Finbarr (1981), Houston and Houston (1989), and Rollins and Rogers (1994). The upcoming CGS publication (White and Greenman, in prep.) will also cover the various mitigation techniques and their use and effectiveness in Colorado in greater detail.

SUMMARY

Collapsible soils are soils that compact or disperse, and induce ground settlement or collapse. There are three different types of soil collapse: mechanical settlement of soil grains, dissolution, and dispersion. Certain geomorphic systems in clay- and silt-rich sedimentary rocks, or evaporite rocks create conditions for the creation of collapsible soil. In mechanical hydrocompactive soil, when the dry, low-density soil with a honeycomb skeletal structure is wetted for the first time, the binding agents of the soil particles or grains break and the soil compacts, inducing settlement of the ground surface. Collapsible soils are prevalent in the semi-arid to arid climatic regions of Colorado. Geographically, that includes the eastern Colorado Plains and piedmont areas along the Front Range, west-central river valleys, and the Plateau areas of western Colorado. In those Colorado provinces there are many occurrences of collapsible soils that have cause structural distress to buildings, roadways, and dams. Though much of the case histories of distress caused by collapsible soils occurred in earlier growth booms of Colorado, mistakes and errors in judgement still continue. Care is needed to properly identify collapsible soils (and the thickness of those deposits) for land-use planning, siting structures, and designing proper mitigative measures for foundations and drainage/landscaping designs.

Figure 11. Distress of recently constructed townhomes (2001-2002) in Glenwood Springs. Note lack of roof rain gutters and nearly flat and poorly drained, rocked-in area below. Bottom left photo shows deflection of doorframe. Middle photo shows deflected beams above hallway as concrete retaining wall on left is settling. Photo on lower right is of doorway shown in middle photo with wall cracks forming as wall is being pulled down in relation to doorframe. Photos from White and Greenman (in prep.) used by permission.

ACKNOWLEDGEMENTS

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EVAPORITE KARST SUBSIDENCE HAZARDS IN COLORADO

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Key terms: evaporite, karst, sinkholes, salt, collapse, dissolution, subsidence

ABSTRACT

Regions of Colorado are underlain by shallow or at-surface rock formations that are composed of evaporite minerals. These precipitated minerals were deposited during the cyclic evaporation of shallow seas that existed in Colorado millions of years ago. The three most important characteristics of evaporite bedrock to engineering geology and land use development are that (1) evaporite rock dissolves in the presence of fresh water; (2) it is generally 10% less dense than normal clastic sedimentary rocks; and (3) it can recrystallize and move plastically or "flow" as a result of differential lithostatic pressure. The dissolution of evaporite rock creates caverns, open fissures, streams outletting from bedrock, breccia pipes, subsidence sags and depressions, and sinkholes. These landforms are described collectively as karst morphology. The term karst originally referred to limestone areas known for characteristic closed depressions, sinkholes, caverns, and subterranean drainage. Evaporite karst comprises similar morphology where these features develop as a result of dissolution of the evaporite minerals. Thin, discontinuous deposits of evaporite occur throughout Colorado in sedimentary formations that were deposited in arid terrestrial and near-shore environments. Certain portions of west-central Colorado along the Colorado and Roaring Fork Rivers, which are currently under high development pressure, lie where massive evaporite formations occur. In addition to evaporite karst, flowage and dissolution of evaporite in these areas has created regional collapse centers. As development continues in areas of evaporite rocks, sinkholes and related ground-subsidence phenomena become geologic hazards, and the potential risks increasingly become engineering and environmental concerns. The Colorado Geological Survey (CGS) is currently conducting a statewide evaporite karst study to address these concerns.

INTRODUCTION

Potential problems with karst environments in Colorado are almost entirely associated with evaporite terrains instead of carbonate rocks. The climate of Colorado is predominantly semi-arid, so carbonate rocks are generally more resistant and form high benches and cliffs. Evaporite rocks, as well as accessory shales and siltstones, weather more easily and generally become the topographic low-lying areas that are more easily developable. Certain areas of Colorado are underlain by evaporite rocks and have experience karst phenomena such as subsidence features, underground water flows, and salinity loading: the most conspicuous and potentially hazardous karst features are sinkholes.

Major areas of evaporite formations, evaporite karst locations, and locations of historic gypsum mining are shown in Figure 1. Four major areas exist in Colorado that contain evaporite-bearing formations: (1) the Eagle Valley Evaporite in west-central Colorado centered around the towns of Eagle, Carbondale, and Buford; (2) the Paradox Formation exposed in breached anticlines in the Salt Anticline Region of southwest Colorado; (3) the evaporite facies of the Minturn Formation of central Colorado in South Park and along the Arkansas River valley south of Salida; and (4) the gypsic Forelle Limestone and related Blaine Gypsum member (Broin, 1957) that exists in the strike valley of the Lykins Formation along the Front Range hogback, from Ft. Collins to Boulder.

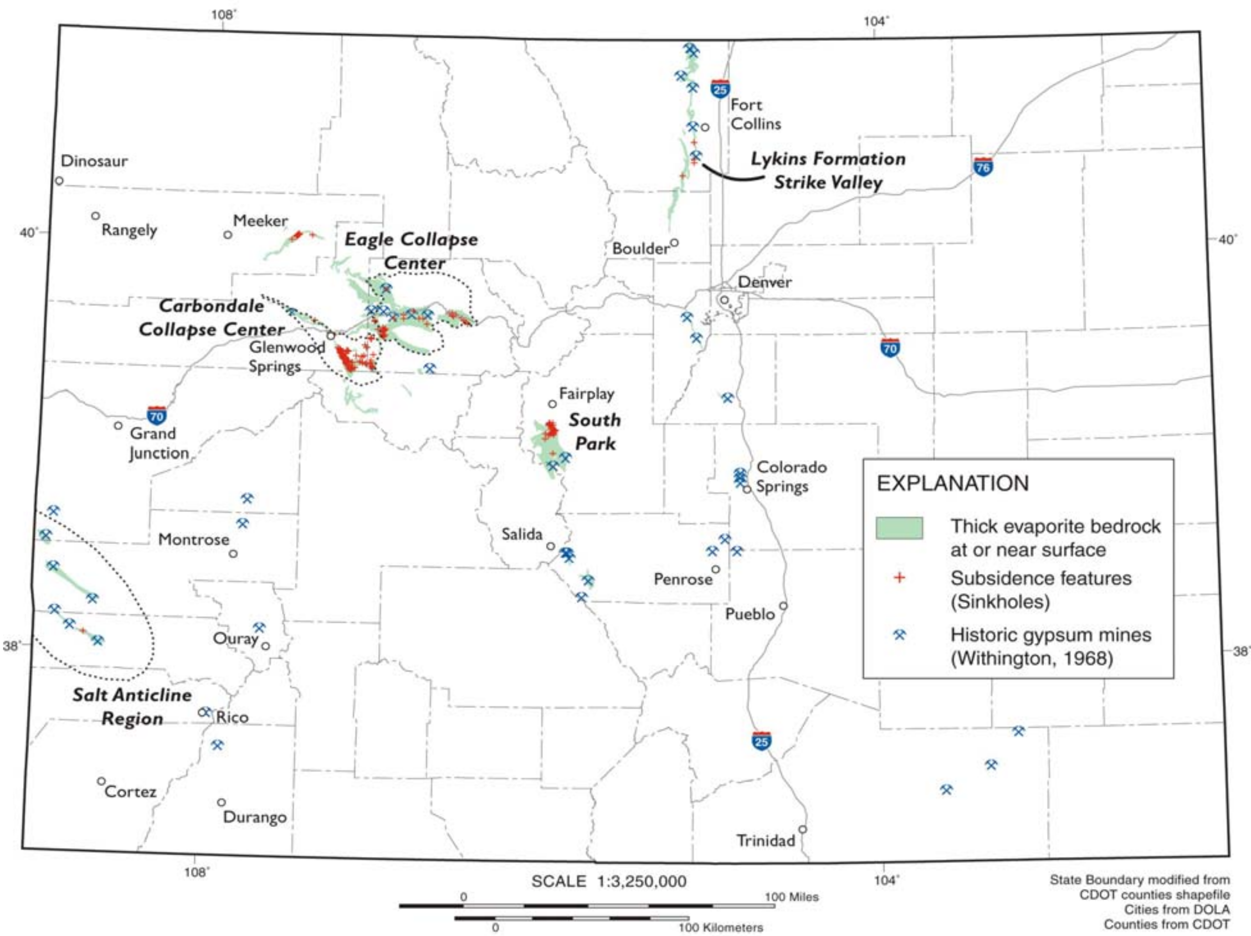

Figure 1. Location of evaporite rocks with approximate thicknesses >20-ft (6.1 m), subsidence features, and gypsum mines in Colorado.

There are other minor areas where thin, discontinuous evaporite beds are exposed, but karst phenomena are not generally present other than minor vugs and small dissolution fissures. Many of these areas have been historically mined for gypsum (Withington, 1968). The Morrison Formation contains gypsum beds that have been mined in Jefferson County and above Penrose in Fremont County, in the Purgatoire River Valley in southeastern Colorado, along the northern walls of the Black Canyon of the Gunnison River, and near Ouray. Small gypsum deposits have also been found in the Hermosa Formation with reported gypsum mining in the Rico mining district along the Dolores River. Small exposure of Lykins Formation gypsum also outcrop near Colorado Springs in El Paso Country and Perry Park in Douglas County. These areas are shown as small mine symbols in Figure 1. Large-scale mining for gypsum continues today near the town of Gypsum where Centex Construction Products, Inc. mines over 500,000 tons per year for use in wallboard and other construction related products. Smaller mines in Larimer and Fremont counties continue to mine small quantities of gypsum for cement manufacture and for soil conditioners (Carroll et al. 2001).

Exciting scientific work has occurred recently in areas of evaporite karst and subsidence in Colorado. The evaporite terrains of the Roaring Fork and Eagle River areas are centered in areas of Neogene deformation and regional collapse, related to flowage, diapiric upwelling, and dissolution of evaporite minerals. Precise geologic mapping, river and hot springs water chemistry, and changes in the superposition (elevations) of various Tertiary-aged volcanic flows has brought about new theories, and definable and defensible limits to areas of regional collapse. Highly contorted strata, collapse debris, structural sag features, deformation of river terrace gravel, piercement structures, and river-centered anticlines are all geomorphic evidence of the subsidence and deformation (Kirkham and Scott, 2002).

While regional collapse related to evaporite flowage and dissolution is fascinating to geoscientists, it is the associated risk from localized and potentially spontaneous subsidence (sinkholes) that can be destructive to facilities and potentially life threatening. Other important considerations include seepage susceptibility, potential failure of reservoirs and dams, and water-quality concerns from dissolved-salt loading of rivers and ground-water wells. The CGS is currently inventorying locations of evaporite karst landforms in Colorado and locations where evaporite rocks are exposed or buried by thin mantles of younger rocks or unconsolidated surficial deposits.

EVAPORITE KARST MORPHOLOGY IN COLORADO

Significant areas of Colorado are underlain by bedrock that is composed of evaporite minerals: predominantly gypsum ($CaSO_4*H_2O$), anhydrite ($CaSO_4$), and halite (rock salt - NaCl). These minerals were deposited during the cyclic evaporation of shallow seas, near-shore desert estuaries and sabkas, and dry terrestrial ephemeral (playa) lake environments that existed in Colorado millions of years ago. As the water evaporated, the remaining solution became hyperconcentrated with salts. Minerals precipitated out of solution and accumulated, creating thick deposits within the basins. Depending on the paleoenvironment, thinly interbedded fine sandstone, mudstone, and black shales can also occur in the evaporite. Subsequent to deposition, the evaporites were buried beneath thousands of feet of younger sediments. They have experienced periodic plastic deformation and flow, particularly during times of mountain building and differential lithostatic loading by erosion and downcutting of river valleys. Over the past several million years, erosion of the uplands and downcutting of the river valley have stripped away the overlying sedimentary rock formations, eventually exposing the evaporite minerals at the ground surface. Near-surface dissolution of evaporite minerals creates subsidence features, called karst topography.

Karst is a topographic term that refers to a type of landform where caverns and open fissures, open subterranean stream flows, closed depressions, and sinkholes exist on, or within, soluble rocks. Evaporite karst refers to topography where these features develop as a result of dissolution of evaporite minerals. Evaporite minerals are generally five times more soluble than limestone (Brune, 1965). In Colorado, there are two fundamental types of evaporite deformation processes: regional subsidence and localized karst geomorphology. They are related but not mutually inclusive. Evaporite karst topography and regional collapse often occur together but evaporite karst can occur were regional collapse (restricted to very thick evaporite deposits) does not.

Most cataloged sinkholes of Colorado occur where suitable thicknesses (generally several meters) of evaporite rock occur. Sinkholes tend to form in unconsolidated surficial deposits such as flat-lying river terraces, recent valleyside sediments, or earlier deposits on pediment slopes overlying the evaporite bedrock. Some sinkholes, fissures, and caverns are exposed in the actual bedrock. In surficial-soil mantles, subsurface borings in the vicinity of sinkholes show wide irregularities of bedrock depths, as do exposures along road cuts. As Figure 2 shows, while the surface of the river terrace is relatively flat, the underlying bedrock surfaces are likely more indicative of karst topography. The highest densities of sinkholes that are manifested at the surface in Colorado occur in (1) the Roaring Fork River-Carbondale area in Garfield County; (2) the Eagle River centered around Gypsum and Edwards in Eagle County; (3) the Buford-North Fork White River area in Rio Blanco County; and (4) Park County south of Fairplay.

Figure 2. Roadway cutslope in mid-Pleistocene river terrace and evaporite rock has exposed dissolution slots and voids filled with gravel near Buford.

Three basic types of sinkholes are found in Colorado (Mock, 2002): (Type A) surface collapse by downward movement of surficial soils into deep bedrock voids; (Type B) surface collapse by piping of fine-grained soil deposits through fissures or small pipes into underlying bedrock voids; and (Type C) spontaneous roof collapse and rubble filling of existing, near-surface dissolution cavities (Figure 3). Sinkholes can also form in formations that overlie evaporite rocks, but only rarely. The sinkholes in disturbed basaltic lava that has collapsed into voids within the underlying evaporite near Colorado Mountain College in the Roaring Fork River valley corridor are good examples.

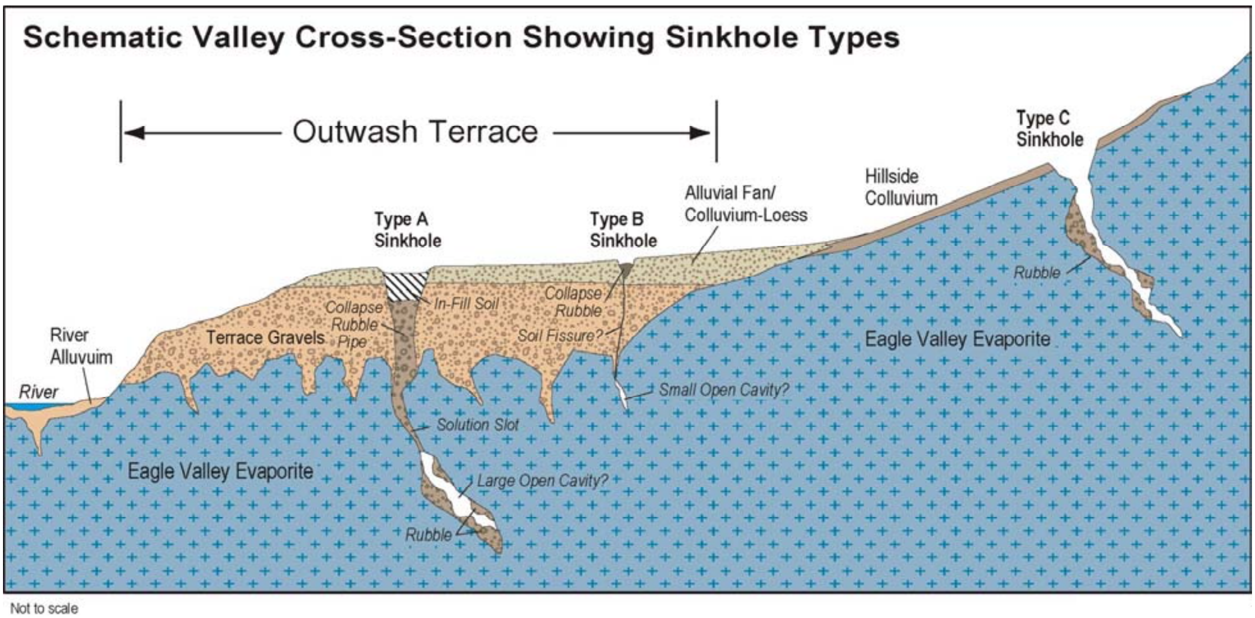

Figure 3. Schematic of sinkhole types (Modified from Mock, 2002).

WEST CENTRAL COLORADO

West-central Colorado contains exposures of Eagle Valley Evaporite. This formation was deposited in a restricted shallow sea basin between the Pennsylvanian/Permian Ancestral Front Range and Uncompahgre Mountains. Massive accumulations of evaporite were deposited that are now exposed along the Roaring Fork, Colorado, and Eagle rivers; and along the North Fork of the White River and behind the Grand Hogback where they are exposed in bands around the White River Uplift (Figure 4). Evaporite karst landforms occur in all of these locations. Significant regional subsidence has occurred in areas underlain by thick deposits of Eagle Valley Evaporite.

Regional subsidence or collapse in the areas involves thousands of square kilometers of crustal deformation, resulting from evaporite flowage and dissolution. Flow can cause localized thinning or thickening of evaporite. When evaporite flow is away from a location, thinning occurs and regional subsidence or collapse of overburden strata or surficial deposits results.

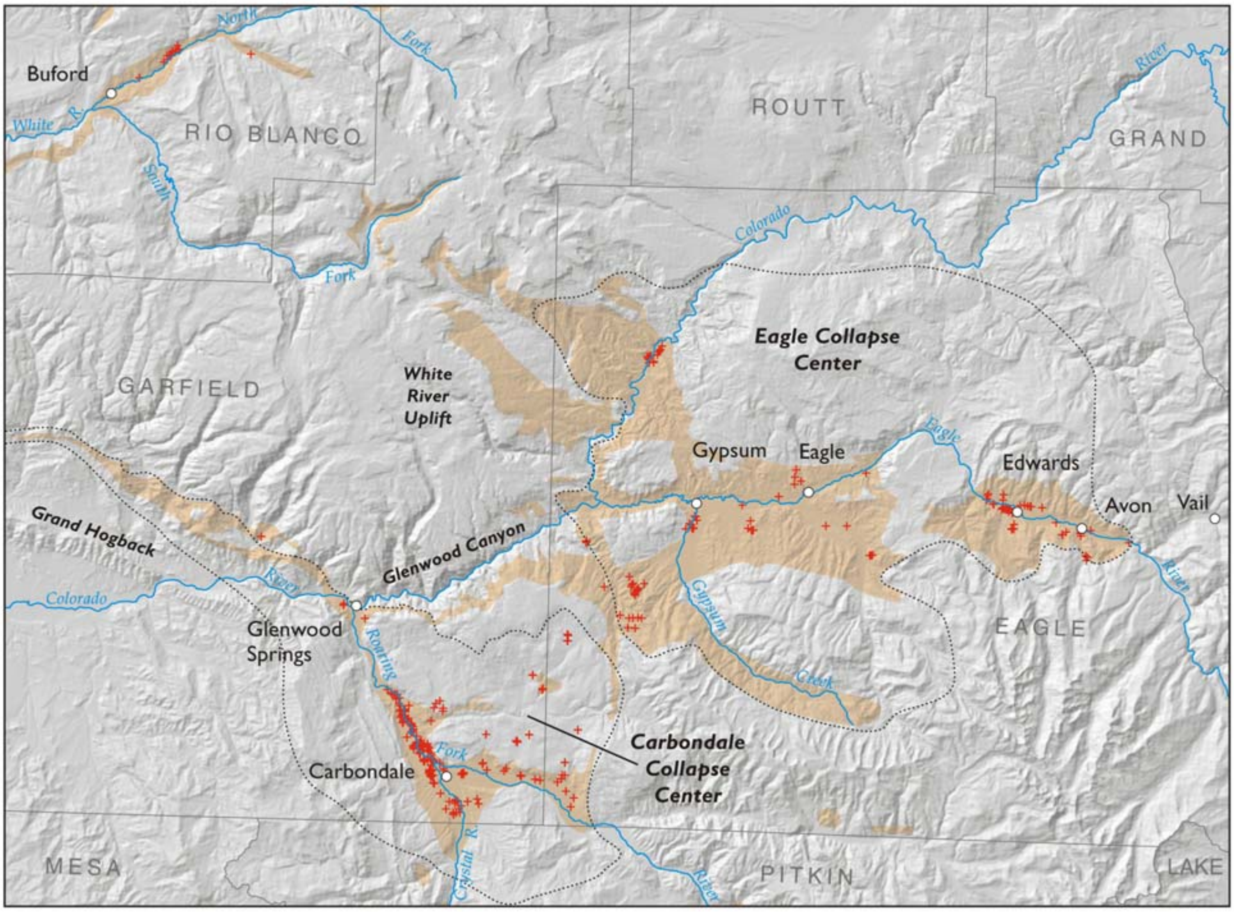

Figure 4. Locations of west-central Colorado evaporite terrain (in tan shading), regional collapse centers, and occurrences of karst features (shown as red crosses).

Thickening occurs where flows occur towards an area. Accompanying such thickening, the lower density evaporite can move vertically and form diapiric uplifts and piercement structures. Kirkham et al. (2001) have shown that such movements of evaporite and regional collapse appear related to differential lithostatic pressures and dissolution. They interpret the cause as Neogene and Quaternary incision of river valleys, and dissolution by both ground water and thermal waters of shallow evaporite where these rock are near the surface.

Two large, adjacent collapse centers have been identified by the CGS and USGS: the Carbondale Collapse Center and the Eagle Collapse Center. The combined area of collapse is at least 1,400 mi^2 (3,600 km^2) and may exceed 1,930 mi^2 (5,000 km^2). As much as 3,900 vertical feet (1,200 m) of collapse is thought to have occurred. The impetus for much of this research was a 1:24,000 scale geologic mapping program. The CGS mapping was conducted along the Roaring Fork and Glenwood Canyon corridors and the USGS mapping was along the Interstate-70 corridor from Rifle to Glenwood Springs and along the Eagle Valley. For more information on the regional collapse phenomenon in west-central Colorado the reader is encourage to review the collection of papers in the recently published Geological Society of America Special Paper 366: Late Cenozoic evaporite tectonism and volcanism in west-central Colorado, edited by Kirkham et al. (2002). Preliminary observations of the Buford area indicate that similar activity, at a smaller scale, is also occurring there. Characteristics of specific corridors in west-central Colorado are further discussed below.

Roaring Fork Corridor

Highly contorted Eagle Valley Evaporite is exposed along the valley walls and floor of the Roaring Fork River. Cattle Creek anticline is a river-centered structure along the Roaring Fork River valley that defines evaporite flow toward the river, where lithostatic pressures are reduced and diapiric uplift results. The photo in Figure 5 shows a diapiric piercement structure where the gray-white evaporite has moved vertically and pierced through the overlying red Maroon Formation. This entire area lies within the Carbondale Collapse Area (Kirkham et al., 2001, Kirkham et al., 2002). The highest sinkhole densities occur between Carbondale and Glenwood Springs along the Roaring Fork Corridor (White, 2002; Mock, 2002). Above Carbondale towards Basalt, large shallow subsidence troughs of up to several tens of acres in size commonly occur on the Roaring Fork River valley floor. The Figure 4 map on a shaded relief base shows the extent of the collapse center and locations of compiled sinkholes and other subsidence features from White (2002).

Figure 5. Diapiric piercement of Eagle Valley Evaporite (light gray formation on left) into Maroon Formation (redbeds on right). Roaring Fork River is in foreground. Meadow and basin areas behind the piercement structure, and below the high hills in the far background, lie within the Carbondale Collapse Center.

The Roaring Fork River is a major transportation and residential corridor between Aspen and Glenwood Springs. As such, there is heavy pressure for development. The traditional ranching economy has been displaced and most of the valley floor is being built out for high-end residential neighborhoods. Sinkhole locations have impacted development plans. Visible sinkholes have been typically avoided during the planning process. Not all subsurface voids have visible evidence of subsidence at the ground surface, however. Rare spontaneous sinkholes occur and, occasionally, during utility trenching or overlot grading, subsurface holes are encountered that were unforeseen (Figure 6). Swarms of sinkholes lie in development areas and as residential density increases the probability of damage to a structure will rise. Figure 7 photos show the density at which sinkholes can occur in the Roaring Fork River valley and the proximity of sinkholes to existing developments. For more information about evaporite-related geologic hazards in this area, see the paper in this volume by Lovekin and Higgins (2003).

Figure 6. Photo on left is a void opened in river terrace gravels from utility excavation in the Roaring Fork Valley. Photo courtesy of Ralph Mock, Hepworth-Pawlak Geotechnical, used by permission. Photo on right is a sinkhole that opened in loess on a soccer field at Colorado Mountain College in February 2003. Maintenance staff has begun backfilling the crater.

Eagle Valley Corridor

The Eagle Valley Corridor runs from Dotsero, at the confluence with the Colorado River, upriver to Avon and the Beaver Creek Ski resort. Exposure of evaporite underlie the Eagle River valley except where the river passes through the Wolcott Syncline, a northward-plunging asymmetrical syncline that exposes the entire sequence of rocks from the Pennsylvanian/Permian Eagle Valley Evaporite to the Cretaceous Mancos Shale. The corridor is centered within the Eagle Collapse Area (Kirkham and Scott, 2002), another regional subsidence area related to evaporite flowage and dissolution.

Karst morphology and related sinkholes occur throughout the Eagle River Valley Corridor (Figure 4). The Eagle Valley is a high-development area. Beaver Creek Resort is near Avon, and Vail is only an additional 15 miles east along Interstate-70. The towns of Gypsum, Eagle,

Figure 7. Swarm of sinkholes in upper photo on mid-Pleistocene terrace. Large sinkhole (full of trash) and smaller sinkholes in the lower photo are on the same terrace about one mile down valley. Note proximity of development on both sides of Roaring Fork River.

and Edwards have become bedroom communities and resort/second home destinations in their own right. Some of the largest sinkholes in Colorado occur in the Eagle River Basin (Figure 8).

There have been several reported occurrences of distress to structures because of subsidence settlement related to sinkhole activity. There have also been circumstances where home excavations have encountered voids and caverns that required backfill or structural solutions.

Figure 9 is a photo of such a location where an in-filled cavern in massive gypsum was found during home excavation.

Figure 8. Large sinkhole near Town of Gypsum. Arrow shows view of photo on right. Eagle Country airport can be seen in right middle background of photo on right.

Figure 9. Home excavation revealing a clay in-filled cavern in massive gypsum. Note small void at top of in-fill, overlain by about 3 ft of gypsum roof below topsoil horizons. Photo courtesy of Steve Pawlak, Hepworth-Pawlak Geotechnical, used by permission.

North Fork of the White River Corridor

The Eagle Valley Evaporite is also exposed on the north flank of the White River Uplift, opposite the Carbondale and Eagle Centers. As seen in Figure 4, a band of Eagle Valley Evaporite is exposed on the Uplift's northwestern flank. This outcrop becomes centered in the valley of the North Fork of the White River near Buford, a small town 25 miles east of Meeker.

The evaporite is exposed along the river floor to a tributary confluence, at which point the Marvine Creek valley follows the evaporite exposure until it becomes obscured by landslide deposits at the base of the Flat Tops, basalt flows that cap the White River Uplift. Where the river valley corridor widens above Buford, a high density of sinkholes exists. One ranch that sits on the river valley here has been historically called Pot Hole Ranch, reflecting the presence of several water-filled sinkholes on the valley floor terrace (Figure 10). The North Fork of the White River valley here is also referred to colloquially as Pot Hole Valley. In 1998 and 1999, the CGS was called upon by the Rio Blanco County Planning Director to evaluate sinkholes that opened within 50 ft (15 m) of a ranch outbuilding on the valley wall of Pot Hole Valley and at the Marvine Ranch Development.

Figure 10. Sinkholes of Pot Hole Valley along the North Fork of the White River, east of Buford.

Water chemistry of the North Fork through the evaporite terrain show higher calcium and sulfate constituents in total dissolved solids, compared with higher carbonate constituents of the South Fork that enters the Uplift area and passes though earlier Paleozoic carbonate rocks (Warner et al., 1984; R. Tobin, USGS retired, pers. comm.). This was not unexpected. Exposures of evaporite overlain by Mid-Pleistocene river outwash terraces show dissolution breccia and solution slots, downwarped gravels, and pipes filled with gravel exposed in roadway cuts (Figure 2).

Preliminary assessments by the CGS indicate that diapiric upwelling, subsidence, and dissolution is occurring in Pot Hole Valley, similar to the Roaring Fork Valley, but on a much smaller scale. During a brief inspection of the valley for the statewide evaporite karst study, it appeared that the river valley, where it widens and contains several sinkholes (Pot Hole Valley), is centered in a shallow anticline. The river-centered anticline, as well as the density of sinkholes and water chemistry changes, is strong evidence that collapse and diapiric movements are occurring; however, the extent of the collapse area is not presently known. More detailed geologic mapping of the area is needed. The area is popular for hunting, fishing, and recreation, and is increasingly being considered for vacation-orientated, second-home and lodge development.

SOUTH PARK

South Park is a large structural basin in the Front Range, mostly lying in Park Country. South of Fairplay and west of Antero Reservoir the evaporite facies of the Minturn Formation, a geochronological equivalent of the Eagle Valley Evaporite of west-central Colorado, is exposed. Figure 11 is a shaded relief map that shows these evaporite and karst occurrences. Most of the

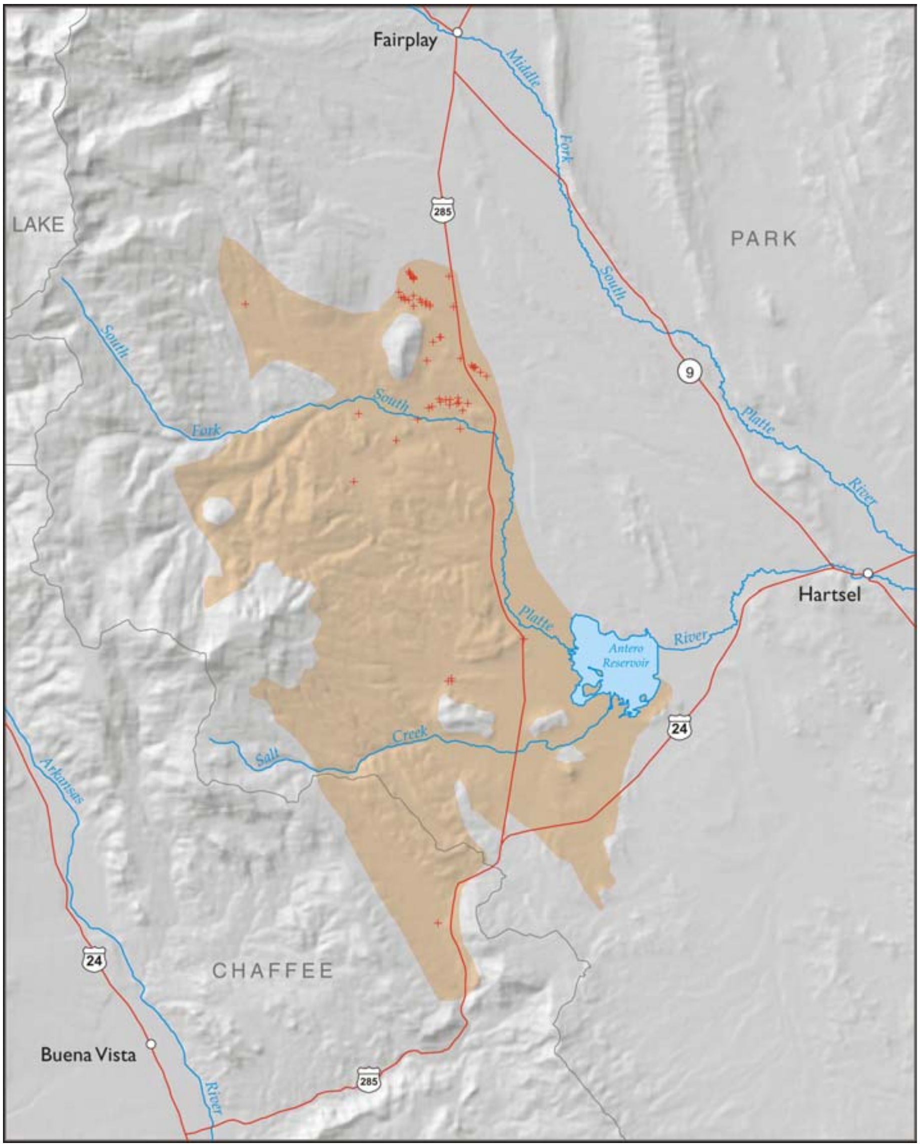

Figure 11. Location of Evaporite Rocks (tan shaded area) and karst features (red crosses) at South Park in Park and Chaffee Counties.

low-lying areas are covered with glacial outwash or recent river alluvium. Evaporite karst landforms are common in this area; sinkholes can be seen from Highway 285, which passes through the eastern edge of it. USGS mapping surrounding a Tertiary intrusion, Black Mountain, revealed over 50 mappable sinkholes (Shawe et al., 1995). The authors reflect that there are likely many others. An interesting side note to this mapping: the work was originally done because the early belief by the authors was that the sinkholes were a cluster of impact craters from a meteorite swarm. Later, they realized that they were karst phenomena. In fact, evaporite karst was recognized in Park Country as early as 1961 by Colorado spelunkers (Davis, 1999).

Davis, in early work from 1961, discussed certain karst landforms in the region: sinkholes and fissures in gypsum rock; the large, 656-ac (265 ha), closed subsidence trough called Long Park; and swallow-holes, subterranean streams, and spring outlets. Davis (1999) spoke of a personal communication he had in 1960 with Mr. Richard McHale, the foreman of the nearby McQuaid Ranch, who said that at irregular intervals, swallow-holes would open in the bed of creeks in the area. These swallow-holes would take the entire creek flow for periods of a few days or weeks until the hole would fill with stream wash and plug off. The Chubb Park area, the portion of evaporite that lies in Chaffee County near Trout Creek Pass, while definitely an erosional park compared to the more resistant limestone and igneous/metamorphic rock ridges adjacent to it, is very broad and flat and may also have a subsidence or dissolution component.

The evaporitic nature of the rocks and resultant salinity of the ground water and surface streams in this area are reflected in some of the topographic names of certain features: Salt Creek, Salt Spring, and Cave Creek are major water features that flow into the South Fork of the South Platte River or Antero Reservoir. Salt Spring (Figure 12) is a subterranean flow that re-emerges at this spring at the base of evaporite hills to meander as a stream into Antero Reservoir.

LYKINS STRIKE VALLEY

The Permian Lykins Formation is comprised of relatively soft, red-colored siltstone and shale with the Forelle Limestone and associated gypsum beds (Broine, 1957). Because it is a soft sedimentary formation, the formation is topographically expressed as a strike valley between the more-resistant ridges of Lyons and Dakota Sandstones along the Front Range hogback. Thicknesses of gypsum have been documented in Larimer County, and to a lesser extent in El Paso County, that were considered economically viable and were historically mined (Withington, 1968).

Evaporite karst has been noted in the Lykins Formation where the gypsum outcrops or is near surface. Of particular interest are the locations of Carter Lake and Horsetooth Reservoirs and the site of the old US Gypsum mine property that has been proposed for annexation by Loveland and is currently planned for development. These locations have had occurrences of sinkholes, and evaporite piping and dissolution.

Figure 12. Salt Spring is a brackish subterranean flow that re-emerges as a meandering stream in South Park. Antero Reservoir can be seen in right middle background.

Carter Lake and Horsetooth Reservoirs, located in the Lykins valley in Larimer County, were built and maintained by the U.S. Bureau of Reclamation (USBR). Carter Lake, based on the early 15-minute topographic map from the early 1900s, is located in a natural subsidence trough. A natural lake existed there prior to the Bureau damming the water gaps and raising the water level. Horsetooth Reservoir was also created by damming several water gaps to form the valley into a reservoir. Active sinkhole formation and accelerated seepage along dam abutments have been of concern to the Bureau for several years now at these two locations. Millions of dollars have been spent recently in investigations and remedial work due to accelerated seepage rates that have been attributed to active dissolution of evaporite minerals (Pearson, 2002). A modification report of Horsetooth Dam (Bureau of Reclamation, 2000) that discusses the geology and seepage concerns is posted on the USBR Great Plains Region website.

Active sinkhole processes have also occurred in the old US Gypsum mineworks near Highway 34 west of Loveland. This potential hazard was discussed during planning for a development called Hidden Valley. Stories circulated in articles in the Loveland Daily Reporter-Herald newspaper that sinkholes had historically opened in the mine workings, and that on occasion, subterranean running waters could be heard from small caverns and fissures. On May 1, 1999, a sinkhole spontaneously opened on the westbound shoulder of State Highway 34 where it crosses the Lykins Formation adjacent to the historic mine property. The Colorado Department of Transportation had to quickly backfill the 25-ft (7.6 m) wide, 20-ft (6.1 m) deep void and their geotechnical group conducted an investigation. The result of their investigation was forwarded to the CGS (Beck, pers. comm.). The City of Loveland is considering annexing the abandoned mine property (i.e., Hidden Valley development) for mixed open space and residential purposes.

SALT ANTICLINE REGION

The Salt Anticline Region is a series of northwest-trending valleys of complex geologic structure in southwestern Colorado. These valleys are topographic expressions of pierced or breached anticlines where evaporite of the Pennsylvanian Paradox Formation of the Hermosa Group has flowed, over geologic time, towards the axis of the anticlines. The resultant morphology, where erosion, dissolution, and collapse have lowered the ground elevation along the axis of the anticlines, is expressed as linear valleys underlain by evaporite rocks. The USGS has mapped most of the 7.5-minute quadrangles of this area and a synopsis of this work is contained in a USGS professional paper (Cater, 1970). Most of the geologic work was a result of heightened interest in uranium prospecting and mining in the 1950s. Cater (1970) mentions the occurrences of sinkholes in these valleys, specifically Big Gypsum Valley, which is underlain by evaporite. However, the descriptions were not specific and he did not include sinkhole locations on any of his geologic maps of the area. The region is isolated and opportunities for development are remote.

The Dolores River passes though the parallel valleys of the Salt Anticline Region in a trellis-type drainage pattern. The water quality work by Warner et al. (1984) shows that a progressive increase in salt loading occurs in the Dolores River as it passes though this area toward its confluence with the Colorado River in Utah.

ENVIRONMENTAL CONCERNS OF EVAPORITE KARST

The major environmental concerns of evaporite karst in Colorado are salt loading of the rivers that pass through the evaporite terrain and unfavorable water quality when water wells are drilled. As mentioned earlier, almost every area where evaporite rocks occur have names such as Salt Creek, Gypsum River, Alkali Creek, Salt Spring, Salt Ranch, Big Gypsum Valley, etc. Several cold and thermal springs with high total dissolved solids exist within evaporite terrain and flow directly into rivers, contributing to the salt loading. The highest point loading of the Upper Colorado River Basin is at the Yampah hot springs in Glenwood Springs. From this one source, 265 tons (240 metric tons) of dissolved halite and gypsum flow into the Colorado River each day (Barrett and Pearl, 1976). Using an average unit weight of 140 pcf (2.23 g/cm^3) the amount of dissolved salts is equivalent to a 141-yd^3 (108 m^3) void every day. At current concentrations rates, this spring alone could account for a cubic mile of evaporite (4.17 km^3) dissolved and washed down the river in 100,000 years. Such quantities put the measured vertical collapse in the west-central collapse centers in better perspective.

Subsurface seepage of saline ground waters also can occur into alluvial aquifers that flow into adjacent river streams. Definite rises in salinity and changes in water chemistry are observed where rivers pass through evaporite terrain (Chafin and Butler, 2002; Kirkham et al. 1999; Warner et al., 1984; M. Sares, pers. comm.; R. Tobin, pers. comm.). Saline seepage may affect water quality and preclude the completion of potable water wells in certain locations. Figure 13 is a reproduction of a graphic from Kimbrough (2001) that shows the pronounced salt loading, expressed as a percent of the total dissolved solids, of water samples taken from the South Fork

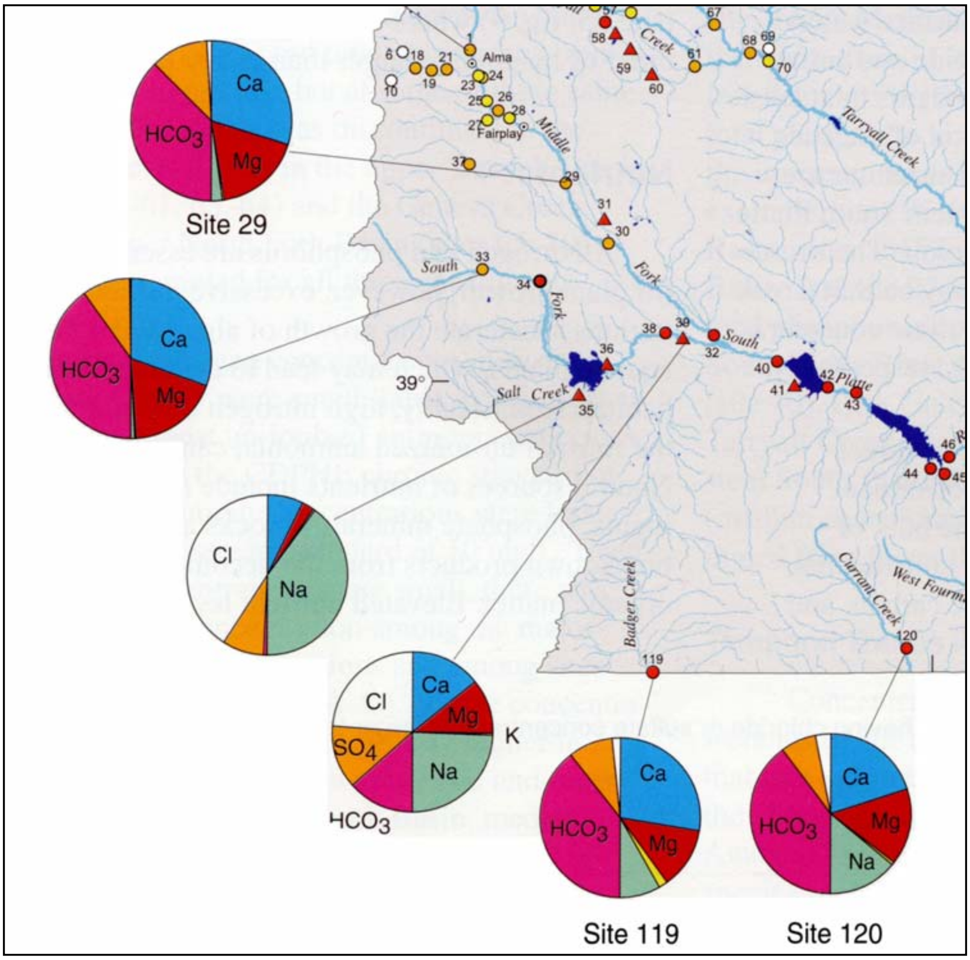

Figure 13. Graphic from Kimbrough (2001) showing major ion percent in sampled stream flows. Note increase in sodium and chloride ion percentages as the South Fork of the South Platte flows through evaporite terrain, and the sample taken from Salt Creek. Refer to Figure 11 for evaporite locations.

of the South Platte River and Salt Creek as they pass through the South Park Evaporite area in Park County.

ENGINEERING CONCERNS WITH EVAPORITE KARST SUBSIDENCE

Where evaporite is exposed at the surface or underlies unconsolidated surficial deposits, karst features occur and there is potential risk that other features, currently hidden, could manifest themselves in the future. Dissolution of evaporite rock creates subsurface voids, chimney rubble and dissolution breccia zones, and fissures. Caverns, open fissures, ground depressions, and sinkholes occur at the surface, all of which can be of concern for development in the area. The hazard is probably greater in areas with higher sinkhole densities; however, future sinkholes may not be restricted to these areas. While spontaneous collapse and openings of subsurface voids can be dangerous and life threatening, such occurrences are relatively rare. They do occur, though. Figure 6 shows two such occurrences. One is a void that was encountered during utility excavation in the Roaring Fork Valley. The other is a spontaneous sinkhole, 25 ft (7.6 m) wide and 20 ft (6.1 m) deep that recently opened at the Colorado Mountain College Soccer field where thick colluvial soils overlie evaporite rocks. More commonly, differential settlement subsidence and removal of fine-grained soils by piping into subsurface voids or fissures, with the resulting differential stress and strain to rigid structures, cause damage to facilities that are unknowingly constructed over or near sinkholes, subsidence troughs, or near-surface underground voids.

Avoidance of known subsidence features is the preferred mitigation alternative, but this is not always possible. Many areas having karst and potential sinkhole and subsidence risks lie in areas of Colorado that are experiencing heavy development pressure, as was shown in Figure 7. The potential risks to these homes have, for the most part, been identified, yet development continues. Because of the high exploratory cost, most home locations have not had investigations to determine the condition of the evaporite rock below the surficial deposits. The typical geotechnical drilling method is augering, either solid- or hollow-flight, which cannot advance through bouldery outwash gravels that are typical of most valley bottoms. Wireline coring, though much more costly and rarely done for residential investigations, can advance though these alluvial deposits and provide cores of the evaporite rocks to better assess their condition.

Many older sinkholes have been covered with recent soil in-filling, or historically filled and forgotten, and are now completely concealed at the surface. Near-surface voids that have not broken through to the surface would also be similarly concealed. Subsurface investigations, either by trenching, a series of borings, or observations made during overlot grading or utility installation, can ascertain whether filled sinkholes and near-surface voids exist within a development area. Low-altitude stereo and oblique aerial photography, eyewitness reports, and historical records may also be helpful to identify filled sinkhole locations. At times, vegetation changes in aerial photography can delineate the boundaries of an ancient sinkhole that is now completely in-filled and not noticeable in the field. There are also geophysical investigation methods that can detect shallow-subsurface voids and soil/rock property changes, such as ground penetration radar, electric resistivity imaging surveys, downhole tomography, magnetic surveys, and seismic surveys.

If sinkholes, near-surface voids, or filled sinkholes are detected and located, an experienced geotechnical firm should be retained to evaluate the hazard and risk potential for future subsidence on the property. On a number of occasions, evaporite bedrock voids have been encountered during excavation of grading and foundation footprints. Figure 9 shows such an occurrence where a clay-filled cavern was encountered during excavation. If this feature had not been fortunately uncovered during excavation, the thin bridge of gypsum would likely fail over time and settlement would result as the bridged material, upon loading and long term wetting, began to settle into the unconsolidated clay. There are ground modification and structural solutions to mitigate the threat of subsidence if avoidance is not an option. Owners and developers should consult with knowledgeable geotechnical and structural engineering firms and ground modification specialty contractors.

Drainage issues and proper water management are important in evaporite karst terrain. Because the bedrock and gypsiferous soils derived from them are soluble, changed hydrologic conditions and increases in fresh water may destabilize certain subsidence areas, rejuvenate older sinkhole locations, or cause new dissolution to occur. Several re-activations of sinkholes have been the result of proximity to irrigation or irrigation ditches. Another concern with evaporite terrain in the semi-arid climates of Colorado is the low-density soils that can be derived from them. Alluvial fan and colluvial slope wash deposits from soft evaporite rocks have been shown to be highly susceptible to hydrocompaction and are prone to collapse-type settlement when wetted. More detailed discussion on gypsiferous soils and hydrocompactive soils are in another paper in this volume (White, 2003).

Subsidence related to Regional Collapse

The modern subsidence rate of regional evaporite collapse areas and the hazard of related ground movements are presently unknown. The risk, while likely very low for current or planned developments (for normal 50- to 100-year residential structures), is also unknown. The rate over geologic time, ranging from hundreds to thousands of years, is significant (Kirkham et al., 2001; Kirkham et al., 2002; Kirkham and Scott, 2002). Figure 14 is a mid-Pleistocene (160,000 BP) terrace in the Roaring Fork River Valley, one of several that have been tilted away from the river because of continued uplift of the Cattle Creek Anticline due to evaporite diapirism. From the work in Kirkham et al. (2001), Kirkham and Scott (2002), Mock (2002), and White (2002) it is apparent that sinkhole formation and dissolution of evaporite continues. Deformation rates related to regional collapse may present undefined long-term risk for development at structural margins where deformation may be highest. This includes areas located near late-Quaternary faults and hinge zones of structural basins, flexural edges and interiors of depressions and sinkholes, and areas underlain by collapse debris.

Figure 14. Deformed and tilted mid-Pleistocene terrace in the Roaring Fork Valley. Note tilt away from center of river valley. The top of this terrace is currently under residential development. White River Uplift is in background. Down valley is Glenwood Springs and confluence with Colorado River in middle background. Photo by R. Kirkham, CGS.

CONCLUSIONS

Evaporite rocks exist at or near surface in many areas throughout Colorado. The dissolution of evaporite rock creates caverns, open fissures, streams outletting from bedrock, breccia pipes, subsidence sags and depressions, and sinkholes: collectively referred to karst morphology. Evaporite dissolution contributes significant salt loading to the Colorado River Basin and, to a lesser extent, the South Platte River. While not covering large areas of Colorado, evaporite rocks underlie certain valleys of west-central Colorado, in addition to an inter-hogback valley of the Front Range. These areas are popular, contain major transportation corridors, and have high development pressures. As development continues and home density in these areas increases, the risk that structures will be impacted by karst subsidence rises. The historic and continuing occurrences of sinkholes, late Quaternary deformation, and salinity loading indicate that active dissolution and regional collapse is still occurring. For these reasons, careful geotechnical investigations and building-footprint siting is needed. Additional work is needed to quantify regional-collapse deformation rates and better determine whether these movements should be considered in the design of structures that lie in these areas.

ACKNOWLEDGMENTS

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MAJOR GEOLOGIC HAZARDS ALONG THE ROARING FORK RIVER NEAR GLENWOOD SPRINGS, COLORADO

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Key Terms: geologic hazards, Eagle Valley Evaporite, calcium sulfate, dissolution, sinkholes, collapsible soils, debris flows, piping

ABSTRACT

The Roaring Fork River valley between Glenwood Springs and Basalt, Colorado, is a major transportation route to the Aspen area and is rapidly becoming an urban corridor. Many types of geologic processes and materials have affected human activities in this area, even before recent development. The authors conducted field investigations for an engineering geologic map of this corridor and studied these processes and materials in detail. The purpose of this paper is to discuss some of the various, potentially problematic geologic conditions. Geologic processes hazardous to engineered works interact and influence each other in complex ways in this region. A common factor, such as the application of water, can bring about more than one hazardous situation simultaneously. Within the study area the dissolution of evaporite minerals, soil collapse, piping, and development of sinkholes are examples of processes driven by water that, in turn, influence each other. The rate of water-driven processes can be increased by water usage in excess of natural moisture conditions. Of special note is the Eagle Valley Evaporite. Many of the geologic hazards identified within the study area are related to the solubility of the evaporite minerals within the formation. Water appears to be the primary causative factor for most of the hazards associated with this formation as well as many of the other hazards identified in the study area. Several of these hazards are particularly noteworthy for the extent of the engineering constraints they impose on the development.

INTRODUCTION

The Colorado State Highway 82 corridor follows the Roaring Fork River valley and connects the town of Glenwood Springs and Interstate 70 to the Aspen, Colorado resort area. Between Glenwood Springs and Basalt, the semi-arid valley remained primarily a rural environment until development increased significantly in the 1990s. Present day building ranges from single-family structures on small-acreage parcels to high-end subdivisions with very large houses, lakes, and golf courses. The bedrock geology includes evaporites, sandstones and fine-grained sedimentary rocks that weather relatively rapidly. Surficial deposits include course-grained stream alluvium and coarse- to fine-grained, gravity and alluvial deposits derived from the local bedrock. Historically, hazardous conditions associated with these deposits and the processes that form them have been ignored, sometimes causing minor to major damage to structures and infrastructure. Many of these geologic hazards are triggered by the wetting of earth materials. Presently, the risk to human activities has increased significantly because most of the modern developments apply significant quantities of water to the ground.

The purpose of this paper is to provide an overview and examples of some of the most significant geologic constraints to development in the area. Many of these geologic conditions and hazards are similar to much of the southwestern United States. Specific discussions of similar combinations of hazards are relatively rare in the published technical literature, so this case study was written to share the authors' experiences in this type of terrain.

The Colorado Geological Survey (CGS) completed geological quadrangle maps (Kirkham and Widmann, 1997, and Kirkham and others, 1996) for the area. As a follow up project, the principal author of this paper conducted an engineering geologic mapping project along the State Highway 82 corridor between Glenwood Springs and Basalt, Colorado (Lovekin, in press). The project was in cooperation with the CGS and partially supported by the U.S. Geological Survey. This paper summarizes some of the findings from that mapping project.

For more information regarding collapsible soil and evaporite karst hazards in Colorado on a statewide basis, see papers by White (2003a, 2003b) in this volume.

LOCATION AND GEOLOGIC SETTING

The study area location is shown on Figure 1. It includes a portion of the Roaring Fork and Crystal River valleys, including their confluence, and is within the Cattle Creek and Carbondale Quadrangles. It lies within the complex geologic transition between the Rocky Mountains and the Colorado Plateau. The surface geology reflects bedrock structure from salt and evaporite intrusions and dissolution (Kirkham and Widmann, 1997). Also, processes of erosion and deposition have formed steep bedrock slopes bounding the river valley and various gravity and alluvial surficial deposits. Topographically the area includes river valleys, tributary drainageways, river and outwash terrace landforms, and steep valley slopes with associated debris fans and rock rubble at the base of these slopes as shown on Figure 2.

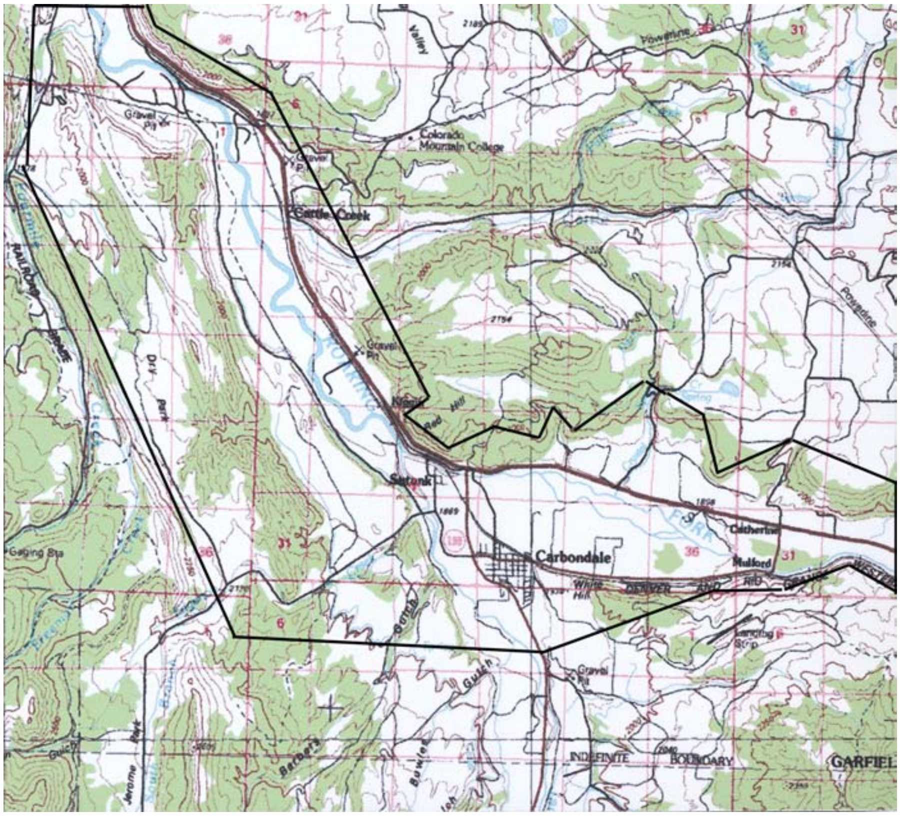

Figure 1. Location of study area.

Figure 2. Typical topographic and geomorphic features of the study area. The Crystal River is in the foreground, its confluence with the Roaring Fork River is just right of center.

Bedrock Formations

The Eagle Valley Evaporite, Eagle Valley Formation, and Maroon Formation underlie the valley alluvium and are exposed on steep slopes on either side of the valley as shown on Figure 3.

Figure 3. Contacts between Eagle Valley Evaporite, Eagle Valley Formation, and Maroon Formations (left to right). The Eagle Valley Evaporite is the white material on the left side of the picture. The Maroon Formation forms the red outcrop at the skyline on the right side of the picture and the Eagle Valley Formation is the material in between.

The middle Pennsylvanian Eagle Valley Evaporite is composed of highly contorted beds of gypsum, and halite and anhydrite (at depth), and mudstone and shale. The highly deformed beds are due in part to diapiric upwelling and the hydration of anhydrite to gypsum as shown on Figure 4. The unit has a vuggy character with cavernous voids up to several feet in diameter and tens of feet deep. The gypsiferous bedrock often weathers into “popcorn”-like soil that is highly susceptible to erosion as shown on Figure 5.

The middle Pennsylvanian Eagle Valley Formation consists of interbedded reddish brown to gray to tan siltstone, shale, sandstone, gypsum and carbonate rocks. Bedrock exposures on slopes typically weather and ravel rapidly and contain abundant rock rubble.

The Permian and Pennsylvanian Maroon Formation consists of maroon and gray-red sandstone, conglomerate, siltstone, mudstone and shale with minor amounts of limestone. Bedrock exposures are usually highly jointed and tend to weather rapidly. Typically, slopes are covered with a rocky rubble that supplies abundant material to erosion gullies and colluvial deposits at the base of the slopes.

Figure 4. Examples of the contorted bedding of the Eagle Valley Evaporite

Figure 5. (A) Popcorn-like weathered surface and erosion channel in the Eagle Valley Evaporite. (B) A weathered surface and dissolution features.

Surficial Deposits

The Roaring Fork River valley bottom is covered with alluvial deposits that interfinger with fans and aprons of alluvial, colluvial, and debris-flow deposits along the valley walls. Predominant engineering geologic map units include (1) river channel, flood plain, and low terrace alluvium; (2) alluvial and debris-flow fan deposits; (3) colluvium derived from steep bedrock slopes; (4) mixtures of colluvium and debris-flow deposits along the valley slopes; and (5) some loess deposits. Geologic hazards associated with each of these deposits are usually related to the material composition or mode of deposition.

Regional Structure

The Roaring Fork River follows the easily erodible Eagle Valley Evaporite, which in the Cattle Creek Quadrangle is also the crest of the Cattle Creek anticline. A salt diapir exists beneath the valley at the junction of the Roaring Fork River and Cattle Creek as shown on Figure 6.

In the 1960s, an exploration well was drilled at the crest of the anticline on the Rose Ranch. The well is reported to have encountered 60 FT (18.3 M) of alluvial gravel, 2,065 FT (629.4 M) of gypsum, anhydrite, and siltstone, and 935 FT (284.9 M) of halite before drilling stopped. The river terrace deposits in this area commonly dip slightly away from the river as a result of diapiric upwelling and subsidence due to dissolution of halite, gypsum, and anhydrite. This has been termed the Carbondale Collapse Center (Kirkham, et al, in press).

Figure 6. The Cattle Creek Anticline is located in the middle of the Roaring Fork River valley (middle background) where Cattle Creek enters the valley (right foreground).

GEOLOGIC HAZARDS

A geologic hazard, as defined by Colorado House Bill 1041 (1974), is "a geologic phenomenon which is so adverse to past, current, or foreseeable construction or land use as to constitute a significant hazard to public health and safety or to property." There are numerous potential geologic hazards in the Highway 82 corridor. Most can be associated with one of three scenarios: 1) interaction of water and evaporite bedrock; 2) interaction of water with weathered bedrock and colluvium on steep slopes; or 3) interaction of water with surficial materials that have a loose soil structure or contain significant quantities of soluble minerals. Since this is a semi-arid region, the natural processes tend to be very slow to progress and the frequency of damage is relatively low. However, recent development in the area is typically adding significant volumes of water to the ground through irrigation systems and concentration of storm water runoff from buildings and flatwork. Based on observations of problems associated with past practices, the new development is likely to result in a much higher than normal frequency of damage to engineered structures unless mitigation measures are put in place.

In the following paragraphs, the hazards will be described with respect to the mechanism of failure, and some local examples will be described to illustrate the resultant damage that can be expected.

Dissolution of Evaporites

Gypsum (a hydrated calcium sulfate) and anhydrite (a calcium sulfate) are the primary minerals in the Eagle Valley Evaporite and are contained, to a lesser extent, in the Eagle Valley Formation. (Anhydrite is usually included in the descriptions of these formations. It is not known if any anhydrite is actually exposed on or near the ground surface or if it has all been hydrated to gypsum). These minerals are highly soluble and have far greater solubility than the calcium carbonate in limestone, which is commonly associated with karst terrain. Brune (1965) indicates that about 2100 parts per million (ppm) of gypsum can be dissolved in typical, non-saline natural water as compared to 400 ppm of calcium carbonate. The solubility of the evaporite minerals is fundamental to the development of sinkholes in the gypsum- and anhydrite-rich bedrock, and can influence several of the geologic hazards related to soils derived from the evaporite bedrock including, collapse, piping, and corrosion potential.

In general, the dissolution mechanism of evaporite minerals in arid climates is complex, and tests for dissolution rates in the field have not been standardized (Hunter, 1993). Soluble evaporite minerals are susceptible to rapid dissolution wherever there is active circulation of ground water that is undersaturated with respect to calcium sulfate. Fresh water from rainfall and irrigation is likely to produce dissolution at a faster rate than ground water that is partially saturated with the evaporite minerals. The fresher the water, the greater the capacity to dissolve evaporites before equilibrium is reached. As the ground-water table is lowered from increased usage, fresh water used for irrigation can percolate to the subsurface and impact areas that were previously in equilibrium.

Dissolution of Bedrock- Within the study area, subsidence thought to result from dissolution of evaporite bedrock can be observed regionally as well as locally. On a regional scale, subtle variations in dip of alluvial terraces overlying evaporite deposits can be observed. An example of this can be seen in the varying dip of the terrace deposits as reflected in the surface topography on Figure 7. This has been explained as the result of the intrusion of the underlying evaporites, and/or the dissolution of the evaporites. (Kirkam, et al, in press).

Figure 7. The surface of these alluvial terrace deposits reflect variations in dip of the deposits. In general, the dip is away from the center of the valley due to intrusion of evaporites, but undulations are likely due to dissolution.

On a local scale, the dissolution of evaporite minerals in bedrock results in the formation of small to large voids and collapsed sinkholes, as a result of natural processes and human activity. An example of sinkholes formed through natural processes is located in the road cut shown on Figure 8(A), where the Eagle Valley Evaporite is covered by alluvial gravels. Figure 8(B) shows a sinkhole in the evaporite bedrock in another road cut that is filled with gravels from the overlying alluvial terrace deposits. Many bedrock outcrops in the study area appear vuggy from

Figure 8. (A) A road cut through the Eagle Valley Evaporite capped by outwash gravels. (B) A sinkhole exposed in the road cut is filled with the overlying gravels.

the natural dissolution process as shown on Figure 9, and voids have been encountered during drilling activities (Mock, personal communication).



Figure 9. Dissolution features in the Eagle Valley Evaporite.

Examples of dissolution of bedrock and the rapid formation of sinkholes as a result of human activities are plentiful on the ground surface of the terrace shown on Figure 7 and 8(A). Reportedly, the area was originally an irrigated potato farm. With the application of the irrigation water, depressions began to appear ranging in scale from 10 Ft (3.1 M) to greater than 100 Ft (30.1 M) in diameter. The depressions disrupted the grade required for irrigation ditches, and the potato fields were abandoned to pasture. The heavy irrigation apparently percolated through the permeable terrace gravels and dissolved the underlying evaporite bedrock within a few to tens of years. The gravels subsequently collapsed into the voids left from dissolution and formed the various sizes of sinkholes illustrated on Figures 10 and 11.

Figure 10. Collapsed sinkholes, seen as depressions in an alluvial terrace, have formed as a result of irrigation-induced dissolution of the underlying Eagle Valley Evaporite.

Another example of sinkhole formation from dissolution of bedrock as a result of human activities is illustrated on Figure 12. A sinkhole collapse (greater than 10 Ft (3.1 M) in diameter, depth unknown) occurred within an unlined irrigation canal at a location where the canal was excavated through Eagle Valley Evaporite outcrops. Apparently, repeated saturation by irrigation waters percolated deep into the bedrock, causing dissolution and formation of the collapsed sinkhole in the canal. As a result, the canal was relocated.

Figure 11. Collapsed sinkholes in the same alluvial terrace, as was shown in figure 10. Sinkholes are partially filled with basalt boulders (A) and trash (B).

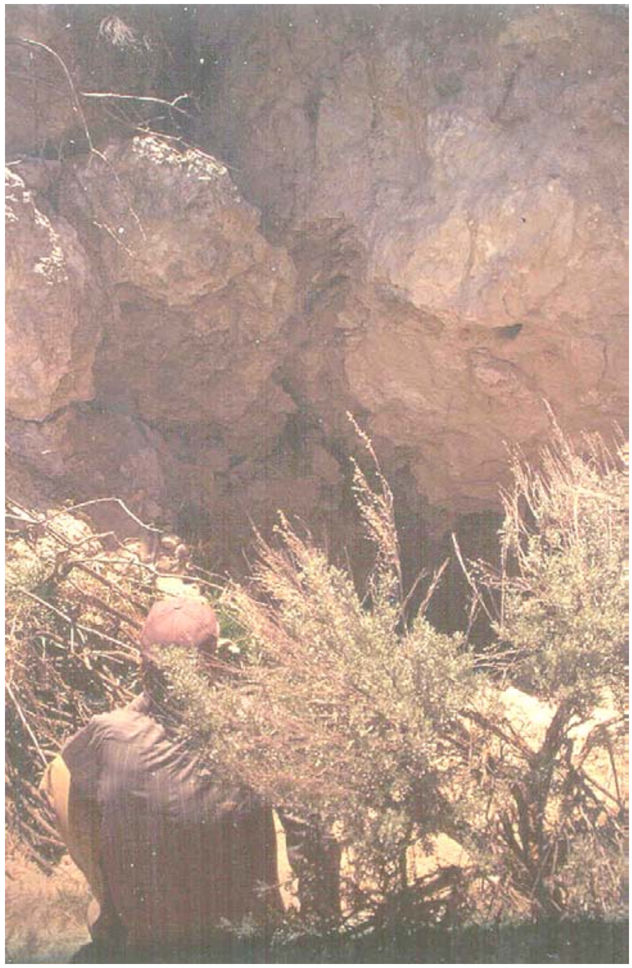

Figure 12. Sinkhole collapse in unlined irrigation canal.

Dissolution of Surficial Materials- Colluvial and alluvial soils derived from the Eagle Valley Evaporite and, to a lesser extent, the Eagle Valley Formation, contain significant volumes of evaporite minerals, primarily gypsum. These deposits are often referred to as gypsiferous soils. The gypsum may be included in the soil as cement or as particles, either of which is susceptible to dissolution in water. The dissolution of soil cement or particles results in a very weak, high void ratio, low-density soil. The result of the dissolution can be collapse of the soil structure and significant subsidence of the ground surface. Some dissolution-susceptible soils in the study area, when subjected to irrigation or concentrated storm-water drainage, have developed a honeycomb structure, which is very weak and can eventually collapse under light loading or possibly its own weight.

As described above, the Eagle Valley Evaporite weathers to a “popcorn”-like soil that is eroded from the steep slopes of the valley walls and transported to the base of the slopes by creep of colluvium and sheet wash. Also, material is transported by running water down small gullies to form small alluvial fans on the valley floor. Typically the drainage areas for these gullies are relatively small, but a significant volume of material is transported to the fans. Even small amounts of rainfall can generate enough runoff on these bare slopes to transport the gypsiferous sediment. Figure 13 shows some of these gullies, small alluvial fans, and sediment aprons at the base of the barren, evaporite bedrock slopes.

Figure 13. Erosion of silty, gypsiferous bedrock and colluvium on steep hill slopes produces gullies that transport fine-grained materials to aprons and alluvial fans at the base of the slope.

The Mid Valley Baptist Church exemplifies problems that can be expected when structures are located on sediments derived from the Eagle Valley Evaporite. The church building is located on a small alluvial fan composed of gypsiferous and silty materials as shown on Figure 14. The fan is formed at the mouth of a small drainage that dissects the steep slope of the Eagle Valley Evaporite. The mouth of the drainage and apex of the fan is within about 25 ft (7.6 m) of the rear of the building. The building foundation was placed partially in this fan and partially on fill material that was derived from the same bedrock. Slowly, the building differentially settled resulting in severe structural damage that became obvious to even the casual observer. Figure 15 illustrates some of the damage.

Figure 14. Mid Valley Baptist Church and the drainages behind it, and the native soils exposed in a trench.

Figure 15. Structural damage showing both lateral and vertical movement within the church building.

When the authors first visited the property they were informed of the damage to the building (Pastor Roland Behnke, personal communication). The structure had settled a total of 20 3/8 in (51.8 cm) from north to south with the south portion of the building settling in relation to the north. Initially, the roof of the building was not guttered and all roof runoff affected the soil adjacent to the building. At one point the pastor took a shovel and was able to push the handle all the way into the honeycombed ground without resistance. Within the structure there was a 4 in (10.2 cm) vertical drift out of plumb over a distance of 10 ft (3.1 m). The floor was pulling away from the walls. The water line had completely rusted out.

Along with the lack of downspouts another source of water that could contribute to dissolution is the natural drainage channel at the rear of the property. Storm-water runoff likely has infiltrated into the fan and migrated under portions of the foundation, dissolving gypsum and causing soil collapse under the foundation load.

Geotechnical investigations concluded that the primary settlement was in the southeast corner and was primarily due to settlement of fill soils from prolonged wetting (Chen Northern, Inc., 1993). Their exploratory boring indicated 9 ft (2.7 m) of loose, clayey sand fill overlying medium-dense, clayey silty sand. Relatively dense, silty sandy gravel with cobbles was encountered from 23 ft (7.0 M) to 31 ft (9.5 m). They note that the fill and the natural soil had high gypsum content. They further note low density and high moisture content in the upper soils. The high moisture content is attributed to continued surface-runoff infiltration. The results of their consolidation testing on a sample taken from a depth of 10 ft (3.1 m) feet indicate a low settlement (collapse) potential. This may be due to previous settlement as indicated by the high moisture contents of the native material (16 to 31 percent). It is likely that a combination of soil dissolution and settlement, both near surface and at depth, are responsible for the total movement observed of this structure.

The congregation considered demolishing the church and building on a new site; however, they decided on foundation repairs that were completed in 1998. White (1998) reported that repair costs were nearly $40,000. Repairs consisted of lifting the foundation and driving piles through up to 52 ft (15.9 m) of dissolution-susceptible soils.

A cabin, shown on Figure 16, is located within a few hundred feet of the church. The cabin is on similar materials and is experiencing similar problems from dissolution of gypsum and/or soil collapse. According to the renters at the time, the cabin was moved to the site and placed on a concrete slab foundation. A lawn was installed in front of the structure and heavily watered. The watering has likely caused both dissolution of gypsum and collapse of the soils. The front of the cabin, the downhill side, has settled considerably in relation to the back of the cabin. This differential settlement has resulted in significant structural damage, which is evident within the building. Extension cracks over 4 in (10.2 cm) wide were viewed inside the cabin. The cabin is shown on Figure 16.

Figure 16. The front (lawn side) of this cabin is settling, apparently due to collapse of soils and dissolution of gypsum from irrigation of the lawn.

The Mid-Valley Kennels are located east of the church and cabin. Standard operating procedures are to hose down the kennels on a daily basis. The slab foundation, which was a year old at the time of our site visit, shows the effects of differential settlement along its expansion joints as shown on Figure 17. Below the kennels, in an area where much of the surface runoff collects, a small sinkhole continually opens up. This is in-filled with gravel on a regular basis. The area of the sinkhole is covered with gravel as shown on Figure 18. Also on Figure 18 is a house, located to the south of the kennel that experiences broken windows on a regular basis from settlement along one side.

Figure 17. Differential settlement along expansion joints at the Mid Valley Kennels where daily hosing of kennels occurs. Flatwork is 1 year old in this photograph.

Figure 18. Re-occurring sinkhole in-filled with gravel below the kennels. The house in the middle distance experiences repeatedly broken windows from differential settlement.

Debris Flow

Steep slopes in the Maroon and Eagle Valley Formations tend to be partially covered with colluvium and rock rubble because of their susceptibility to rapid weathering. The slopes are dissected by numerous, small drainages that are the source of occasional flooding, hyperconcentrated flows, and debris flows as shown on Figure 19. The drainage areas tend to be small, but due to shallow soil and sparse vegetation, runoff can be significant during rainstorms or rapid snowmelt. At the mouth of these drainages, debris fans coalesce and mix with rocky colluvium as shown on Figure 20. Recent debris flow deposits in the upstream portion of the fan, as shown on Figure 21, show that this is an active process. These deposits caused the Aspen Glen subdivision to decide not to develop this portion of their property because of the elevated risk of damage from debris flows.

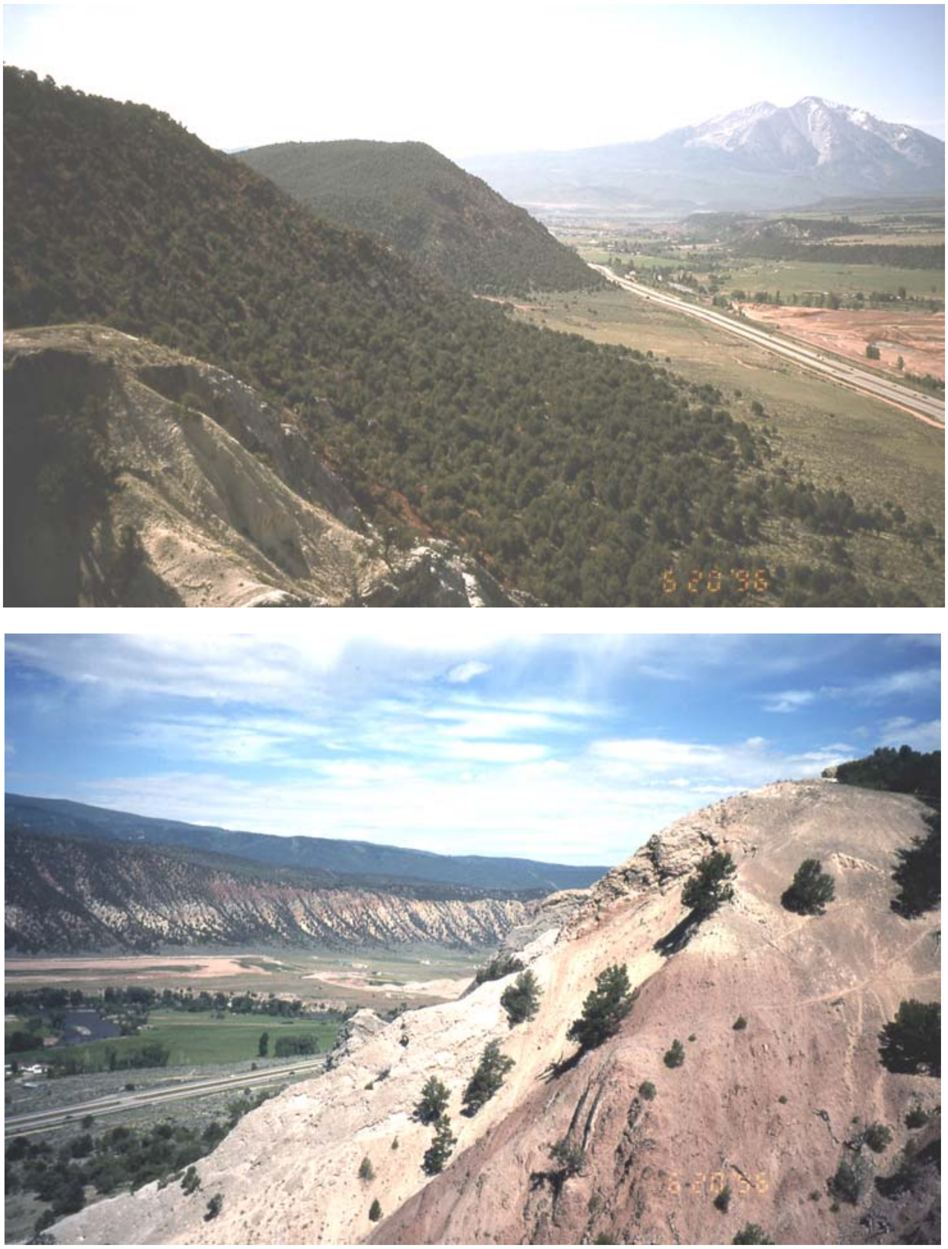

Figure 19. Steep slopes dissected by numerous small drainages form the source areas for debris flows. Debris fans coalesce and mix with rocky colluvium at the slope base.

Figure 20. (A) Coalescing debris fans along the base of steep slopes. Note the vegetation change along the distal edge of fans. (B) Recent debris flow deposits on a fan.

Variation in material texture on the debris fans is illustrated on Figure 20A. The predominately coarse-grained fraction of the deposit extends from the channel mouth to the margin of evergreen foliage, and a dominately silty, well drained material extends to the edge of the sagebrush. Typical of debris flow deposits, the coarse-grained clasts are supported by a silty matrix. Running water carries additional silt farther down the fan. The rapid deposition and drainage of these deposits leaves a high void ratio, low-density structure in which clasts are supported by a dry silt and clay binder. This type of structure may be in a meta-stable condition in a semi-arid

Figure 21. Recent debris flow deposits on upstream portion of a fan. (B) Debris flow natural levee near mouth of drainage and apex of fan.

climate. The stability of the structure can be affected by the later addition of water to the deposit, either naturally or by irrigation.

In July, 1998, a debris flow inundated a county road in the study area. This flow occurred in different geologic materials than described above and is an example of how human activities can trigger geologic processes in this terrain. The debris flow originated as a slope failure in a steep, alluvial terrace slope (the same terrace as shown on Figure 8(A)). The alluvial gravels overlie Eagle Valley Evaporite bedrock. The Crystal River Ranch uses the terrace for pasture and irrigates it to maintain sufficient grass to support livestock in dry months. Individuals near the ranch reported that irrigation remained running much longer than normal on the area above the slope where the debris flow initiated. Examination of the scarp in the terrace slope face, shown on Figure 22(A), suggests that the debris flow initiated by a sudden slope failure of the gravels along the contact with bedrock. The resulting debris fan is shown on Figure 22(B). It is likely that the heavy irrigation, in combination with higher than normal precipitation, built up a high ground-water head in the gravels and initiated the sudden slope failure. Following the slope failure, the gravels rapidly dewatered and moved down the existing drainage as a debris flow. White (1998) described the incident in some detail and suggested a possible failure scenario. He estimated approximately 65,000 yd^3 (49,696 m^3) of material was involved in the slope failure and debris flow.

Figure 22(A) Landslide scarp in the steep slopes of an alluvial terrace (B) the resulting debris flow deposit. Figure 22(B) is by Jon White (CGS) and Robert Florez (CDOT Aerial Reconnaissance Group) and was used by permission.

Collapsible Soil

Collapsible soils generally are fine-grained deposits with a meta-stable structure that have never been fully saturated with water. Deposits containing coarse materials supported by a fine-grained matrix can be susceptible to collapse as well. The structure of the soil is typically a combination of silt grains, clay, and in the study area, sometimes evaporite minerals. The silt grains are bound to one another by the clay and/or evaporite minerals and form an open structure (high void ratio and low density). The deposit maintains the open structure because the cementing material has a relatively high dry strength (Barden, et al, 1973), which prevents consolidation under increasing overburden stresses such as subsequent deposition and/or surface loading from structures. Collapsing soils are common where rainfall rarely penetrates below the root zone (Mulvey, 1992).

This situation is typical of semi-arid to arid environments such as the study area. Moisture deficiency at or immediately following the time of deposition is the main criteria for the deposit or soil to be prone to collapse. The collapsible units are typically deposited as wet slurrys that drain rapidly, not allowing consolidation of the soil structure.

In deposits formed by flash flooding and debris flows, collapsible strata may be quite thick or located at depth as well as near the ground surface. In dry climates, saturation of these deposits may not occur naturally very often, so that the open soil structure may be preserved for long periods of time. This type of void structure is typically macroscopic, that is, visible to the unaided eye. Upon inundation with water, these deposits undergo sudden changes in structural configuration with an accompanying decrease in volume (White, 2003).

A variety of terms have been used to describe this process including hydrocompaction, hydroconsolidation, collapsible soil, soil collapse, settlement, shallow subsidence, and near-surface subsidence. The literature on this subject generally favors the use of the term "collapsible soil" (Dudley, 1970; Waltham, 1989) to refer to deposits susceptible to sudden volume decrease upon inundation with water.

The application of water causes the meta-stable structure to destabilize and the mechanism is twofold. First, the large void volume allows water to fill the deposit beyond its liquid limit. Second, the structural bonds of clay (lowered cohesion) and evaporite minerals (dissolution) are destroyed by the presence of water that causes dispersion and shear failure of the bonds. The soil structure thus “collapses” into a more stable, denser configuration. This type of volume decrease occurs with no change in vertical load and is due solely to the effects of water. The soil collapse is sudden and may cause up to several feet of surface subsidence depending on the thickness of the collapsible unit. Up to 10 percent volume reduction may be experienced in these deposits. This much subsidence has obvious implications for engineered works.

Loess deposits are often susceptible to hydrocompaction or collapse when wetted. Loess is a wind-deposited soil composed primarily of silt-sized particles. It is characterized by a loose structure that consists of silt and fine sand particles coated by a clay binder (Higgins and Modeer, 1996). The effects of wetting on the soil structure may be similar to those described above.

Ralph Mock (personal communication) reports that these potentially collapsible deposits typically have a low plasticity index of about 6 percent. Within the study area the deposits of colluvium, loess, and alluvial fans all can have some collapse potential. The soil consolidation test is the most commonly used method for identifying the collapse potential of a deposit. The test measures sudden compression of a sample upon wetting under constant load.

Hepworth-Pawlak Geotechnical (Mock, personal communication) has characterized the collapse potential of the soils in the study area based upon results from the one-dimensional consolidation test. The following table summarizes the collapse potential rating based on the results of the one-dimensional consolidation test.

Collapse Potential	Percent Compression from Consolidation Test
Non-collapsible	<1%
Low	1% to 3%
Moderate	3% to 5%
High	>5%

Figure 23(A) shows a collapsed area in silty soils near the downstream edge of a debris flow fan. The source area for the fan is the Maroon Formation. This same area had largely been in-filled by sheetwash in the intervening 15 or so years as shown on Figure 23(B).

State Highway 82 and various county roads commonly experience settlement problems when the routes cross debris flow fans. The highway crosses numerous debris flow fans between Glenwood Springs and Carbondale. Over time differential settlement from collapsible soils disrupts the rather flat grade, which results in the need for regrading and paving more frequently than normal.

Piping

Piping is a subsurface-erosion process whereby fines (primarily silt- and clay-sized particles) are entrained as water flows through a soil. The boundary between pervious and poorly drained soils occurs approximately at 3.9×10^{-5} in/s (10^{-4} cm/s) (Costa and Baker, 1981). If the soil permeability is equal or greater than this value, it may be susceptible to migration of particles in ground water, which can result in "pipes" being formed. Over time the pipes may enlarge and eventually collapse.

Figure 23(A) Pat Rogers in a collapse feature (Shelton and Prouty, 1979). Figure 23(B) The same feature about 15 years later.

Silt-rich deposits within the study area are known to erode by piping, as shown on Figure 24. Piping has been observed in colluvium, eolian (loess), and alluvial deposits in the area. It may be initiated by heavy irrigation or by ponding of water. Much of the piping damage observed was associated with human activity. Flow from heavily irrigated fields may percolate laterally in the subsurface and daylight in a road cut or hill slope at the edge of the field. Piping may also start in root holes or animal burrows. Several examples were observed of greatly enlarged pipes forming or multiple pipes coalescing and collapsing as shown on Figure 25. Typically this results in minor to moderate erosion damage. In a populated area, the opportunity exists for piping to occur under structures and to result in a potential collapse of the pipe and damage to the structure. Piping may be severe along leaking or ruptured water lines. In such settings, pipes can form and expand rapidly and result in very large voids that can undermine foundations.

Figure 24. Piping derived voids in silty, surficial deposits.

Figure 25. Piping erosion in slope below irrigated field.

Along the Highway 82 corridor another form of piping is relatively common. In some cases, the piping of silty deposits is related to the dissolution of evaporite minerals. Where these silty deposits overlie evaporate bedrock that contains voids, heavy irrigation or ponding of water may cause the soil to pipe from the ground surface into the voids.

In areas of silty soils, any surface water use that is in excess of natural rainfall should be closely controlled to avoid piping. Water impoundments or irrigation ditches require lining to avoid failure by piping. Water lines through piping susceptible soils should be fitted with seepage collars to reduce the risk of leaks that could lead to large voids.

Corrosion

Known weathering by-products of gypsiferous soils and bedrock include sulfuric acid and sulfates (Bell, 1981). These by-products are corrosive to Portland cement, concrete, and metals. When these materials are in contact with the soil, bedrock, and/or ground water, their service expectancy is typically shortened significantly. Individual homeowners describe extensive corrosion to their water-well systems that requires premature replacement after only some years of use. This includes the owners of the house shown on Figure 18 as well as the Mid Valley Baptist Church.

CONCLUSIONS

An engineering geologic mapping project along the Colorado Highway 82 corridor between Glenwood Springs and Basalt, Colorado, identified numerous significant geologic hazards. Most of the hazards can be associated with at least one of the following conditions: 1) interaction of water and evaporite bedrock; 2) interaction of water with weathered bedrock and colluvium on steep slopes; or 3) interaction of water with surficial materials that have a loose soil structure or contain significant quantities of soluble minerals.

The addition of water to evaporite bedrock tends to form solution voids that may collapse suddenly and damage structures. Water flowing over the steep, weathered slopes may initiate debris flows that travel well onto the fans at the base of the slope. The application of water to the loose, fine sediment on debris and alluvial fans, or sediments derived from evaporite bedrock, can cause collapse of the soil structure and potentially damaging surface subsidence. The application of water to evaporite bedrock and soil forms a corrosive environment for concrete and metals.

Recent development is typically adding significant volumes of water to the ground through irrigation and/or concentration of storm-water runoff from buildings and flatwork. Increased population and water is likely to result in a much higher than normal frequency of damage to engineered structures, unless potential hazards are identified prior to building that would allow avoidance or mitigation measures to be put in place.

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COLORADO'S FRACTURED, CRYSTALLINE-ROCK AQUIFERS

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Key Terms: crystalline-rock aquifer, fracture, hydrogeology, ground water, wells, well yield, igneous rocks, metamorphic rocks

ABSTRACT

Colorado's crystalline rocks consist of Precambrian age igneous and metamorphic rocks and Tertiary age igneous rocks. These rocks are exposed at the surface within the Southern Rocky Mountains physiographic province that occupies the western central portion of the state, and represent the fractured, crystalline-rock aquifers that supply much of the domestic water supply needs in the mountainous portions of Colorado. With no primary porosity, water storage and flow in crystalline-rock aquifers occurs from fractures in the rocks. The intensity and width of fractures and the degree of connection between fractures influence the water storage characteristics of these aquifers. As the fracture porosity of the crystalline rocks is generally less than 1%, these aquifers are incapable of storing significant quantities of ground water. Data from the Colorado Division of Water Resources well-permit database and published studies are used to present the distribution, depth, water level, and yield of wells completed in the fractured, crystalline-rock aquifers. In response to population growth pressure, tens of thousands of small-diameter wells have been drilled in the mountainous regions of Colorado to accommodate domestic and some limited public water supply.

INTRODUCTION

Colorado's crystalline rocks consist of Precambrian age (950 to 1,800 million years old) igneous and metamorphic rocks; largely granites, gneisses, and schists; and Tertiary age (less than 65 million years old) volcanic and igneous intrusive rocks. Unlike sedimentary rocks, igneous and metamorphic crystalline rocks have no primary porosity. As such, these rocks represent a unique and expansive aquifer system where water is stored in fractures. As an overall percentage of the total rock volume, these fracture spaces are very small (<1%). Consequently, the water storage capability of Colorado's fractured, crystalline-rock aquifers is low. This characteristic of the aquifer is often a limiting criterion for proposed development in the mountainous regions of the state when ground water resources are utilized for water supply. Crystalline rocks occupy approximately 19 percent of the state's total area, and much of the domestic water supply needs in the mountainous portion of Colorado.

Crystalline rocks are exposed at the surface throughout the Southern Rocky Mountains physiographic province that occupies the western central portion of Colorado (Figure 1). This province is characterized by several distinct mountain ranges with elevations ranging from 6,000 to over 14,000 ft (1,830-4,270 m). The individual mountain ranges are separated by valleys and high intermontane parks such as North Park, South Park, and the San Luis Valley (Figure 1). The eastern edge of this province follows the slope breaks of the Front Range, Rampart Range, Wet Mountains, and the Sangre De Cristo Mountains. The western border is more irregular encompassing portions of the San Juan Mountains, Elk Mountains, Sawatch Range, Gore Range, White River Uplift, and the Park Range (Figure 1).

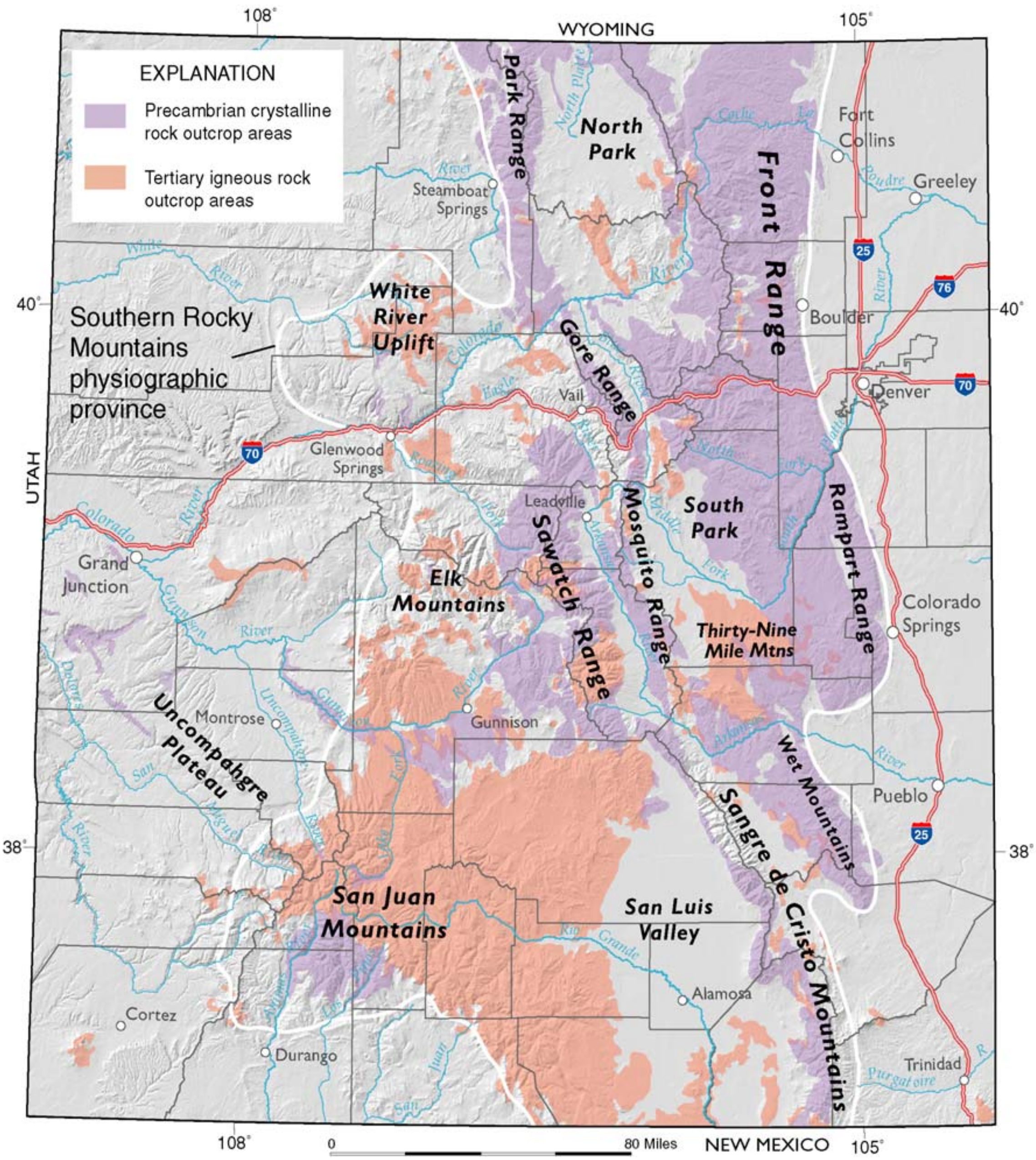

Figure 1. Geographic features and crystalline rock outcrop areas in Colorado.

General features of this province include high peaks, great relief, rugged terrain, steep slopes, shallow soils, and extensive areas of exposed bedrock. The mountainous portions of Colorado are also endowed with mineral wealth and fabulous scenery. Vegetation varies from alpine tundra above timberline to thick stands of evergreen forests with interspersed communities of aspen trees. Scrub oak is common at lower elevations, particularly on the south-facing slopes. Mountain meadow vegetation is primarily grasses and wildflowers.

The Precambrian igneous and metamorphic rocks and the Tertiary igneous rocks are discussed together because of their close proximity in the mountainous area of Colorado and their hydrogeologic similarities. Metamorphic rocks, primarily gneisses and schists of metasedimentary and metavolcanic origin, represent the major rock type in this province and are the oldest of the Precambrian rocks. Younger Precambrian igneous rocks, primarily granite, have intruded the metamorphic rocks in the form of batholiths, plutons, and dikes. The Tertiary age intrusive and extrusive (volcanic) igneous rocks generally lie west of and between the outcrops of Precambrian rocks (Figure 1).

As crystalline rocks are typically in areas of rugged terrain and higher elevations, the population is sparse except along the western edge of the Denver-Boulder metropolitan area; in and around old mining towns such as Leadville, Silverton, Blackhawk, and Cripple Creek; and in popular ski area towns. With the exception of the metropolitan area along the Front Range, the population density in the mountainous regions of Colorado is 39 or fewer residents per square mi (2.59 km^2). According to the 2000 Census, the central mountains of Colorado represent one of the fastest growing subareas of the state at nearly 42 percent growth (U.S. Census Bureau, 2000). Land use is primarily forest, with the majority of crystalline rocks outcropping within National Forest boundaries. Small parcels of the National Forest land are permitted for ski areas, but much of the National Forest is unpopulated except for small inholdings of private land.

Primary industries in the areas of crystalline rock are logging, mining, and tourism. In 2001, 140 million board ft (330,422 m^3) of timber were harvested from the National Forests in Colorado. While mining has declined considerably in importance in the last 50 years, a number of mines are still operating. Most of the mining is for gold and silver, but other precious and base metals are also mined. Outdoor recreational opportunities including skiing, mountain biking, hiking, hunting, and camping, among others, abound in the high country attracting tourists nationwide.

Since the source of ground water is infiltration of precipitation or snowmelt, rates of precipitation, evapotranspiration, and runoff all impact ground-water resources. Average annual precipitation in the high mountainous regions of Colorado generally exceeds 40 in (100 cm), reaching a maximum of 60 in (152 cm) in the high peaks of the Park Range northeast of Steamboat Springs (USDA and NRCS, 1999). Average annual lake evaporation rates range from under 35 in (89 cm) in the central and western portions of the state to 45 in (114 cm) along the eastern edge of the Front Range (Farnsworth, et al., 1982). Average annual runoff ranges from less than 5 in (13 cm) along the Front Range to 20 in (51 cm) or more at the headwaters of the major rivers, where slopes are steep and there is considerable bedrock exposed (Robson and Banta, 1995). The mountainous portion of Colorado is the only area in the state with a potential surplus water balance, generally between 4-8 in per year (10-20 cm/yr) (Waltman, 1997).

FRACTURED, CRYSTALLINE-ROCK AQUIFERS

Within the Precambrian rocks, investigators have identified three hydraulically significant subgroups: (1) metamorphic rocks dominated by gneisses and schists, (2) igneous intrusive rocks such as granites and quartz monzonites, and (3) major fault zones that cut both rock types. All of the lithologies of subgroups (1) and (2) are jointed (fractured), and this system of joints forms the ambient or background permeability of the aquifer. In addition, these rocks and their associated joint networks have been subjected to brittle deformation producing fault zones of varying styles. The tectonic forces associated with repeated uplift of the Rocky Mountains have produced the complex joint and fault patterns that are observed in the rocks today.

These fault zones along with intruding pegmatite dikes have much higher fracture densities, making them spatially complex conduits for ground water. The productivity of a crystalline aquifer is very dependent upon the location and geometry of brittle fault zones. Investigators have been split regarding evidence to suggest one rock type is more productive than another. While fracturing can be relatively uniform in igneous rocks, the compositional heterogeneities in metamorphic rocks have a large influence on fracture spacing, intensity, and orientation.

Over most of their outcrop area, a thin veneer of surficial deposits with moderate to high permeability generally less than 8 ft thick overlies the crystalline bedrock. The surficial deposits are typically not extensive enough to yield suitable quantities of water, but are an important unit for recharge and shallow, seasonal ground-water discharge. A conceptual model of the fractured, crystalline-rock aquifer system is shown in Figure 2. The intensity and width of fractures and the degree of connection between fractures influence the water storage characteristics of the aquifer, with higher values occurring where there are relatively wide, interconnected fractures. This network of fractures and joints in crystalline rocks is exemplified by the photograph in Figure 3.

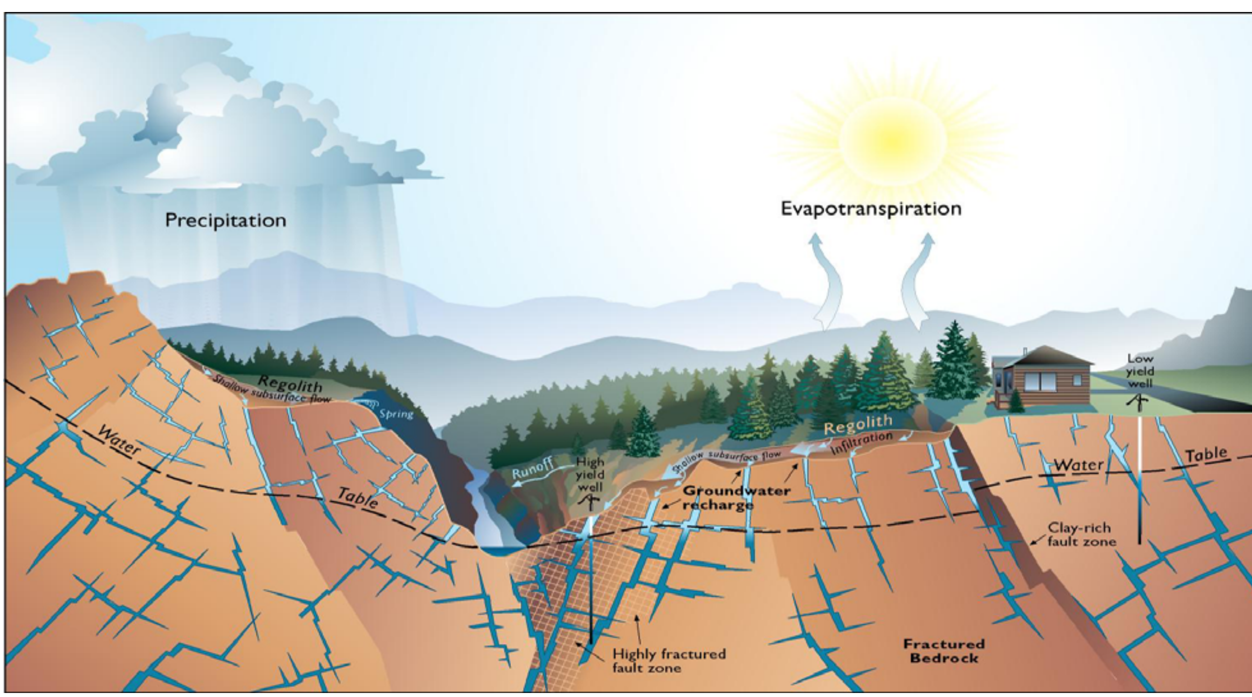

Figure 2. Conceptual model of the fractured crystalline-rock aquifer system.

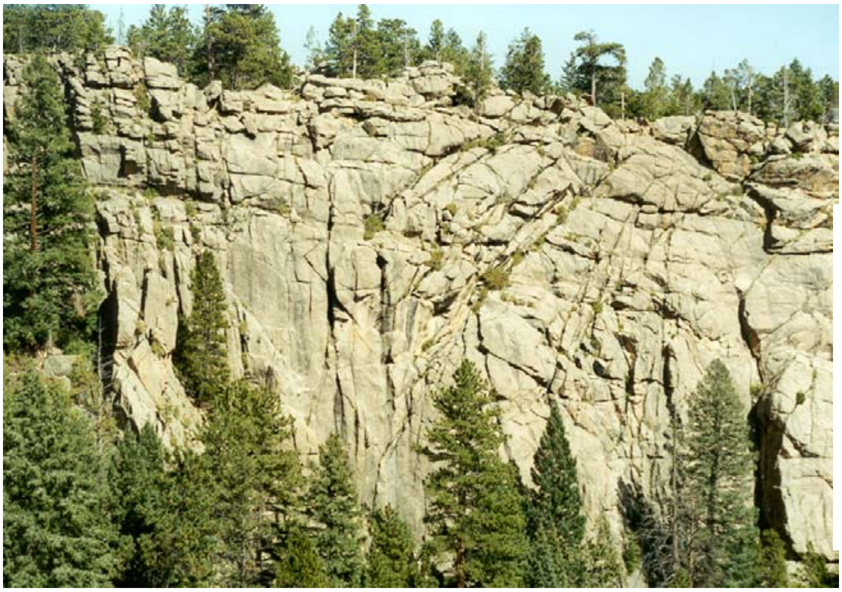
Photo by J. Caine, USGS

Figure 3. This granitic cliff in western Jefferson County exemplifies the variety of fracture orientation and density.

In general, ground water within the fractured, crystalline-rock aquifers is unconfined with water levels fluctuating seasonally and correlating with precipitation events. Recharge to the fractured, crystalline-rock aquifers is predominantly by infiltration of precipitation and snowmelt. The predominant recharge is from snowmelt occurring between the middle of May and the first part of July. Water levels can fluctuate ten's of feet depending on the season and yearly variations in precipitation. In general, water levels are highest in the spring or early summer when there is high runoff, and lowest in the winter when frozen ground and precipitation in the form of snow rather than rain inhibit recharge.

Robinson (1978) reports the amount of recharge depends on the amount of snow, its moisture content, and the rate of melting, as well as on the infiltration characteristics of the surficial materials, the rate of evaporation from the surface, and the rate of transpiration of the vegetation overlying the bedrock. Recharge may also occur where fractured rock underlies saturated alluvium, water storage reservoirs, or individual septic disposal systems. Recharge to any given well generally occurs in the immediate vicinity of the well. Florquist (1973) found that in most granitic rocks, the recharge for a given well is within 200 yards (183 m) of the well.

Recent studies indicate that some 85 to 95 percent of precipitation is returned to the atmosphere by evapotranspiration (Litke & Evans, 1987; USGS, 2001). Only a fraction of the remaining water enters the ground-water storage system, which includes the surficial soil layer. This limited amount of recharge to the aquifer suggests that a delicate balance exists between aquifer recharge and consumption in the more populous regions of Colorado's high country.

Regionally, the water table mimics the surface topography. The general flow direction is downslope and toward surface drainages. Discharge from crystalline rock aquifers occurs at natural springs, as baseflow in adjacent stream drainages, and by ground-water withdrawal from wells.

DISTRIBUTION AND HYDROGEOLOGIC CHARACTERISTICS OF PRECAMBRIAN ROCK AQUIFERS

The extent and distribution of the exposed Precambrian crystalline rocks is shown in Figure 4. These rocks occupy about 12 percent of the surface area of Colorado (Tweto, 1980). Precambrian-cored mountain ranges in north-central Colorado include the Park Range east of Steamboat Springs, the Gore Range to the south of the Park Range, and the Mosquito Range east of the upper Arkansas River (Figure 1). Precambrian rocks are typically classified on the basis of mineral composition, such as biotite gneiss, sillimanite schist, and quartz monzonite.

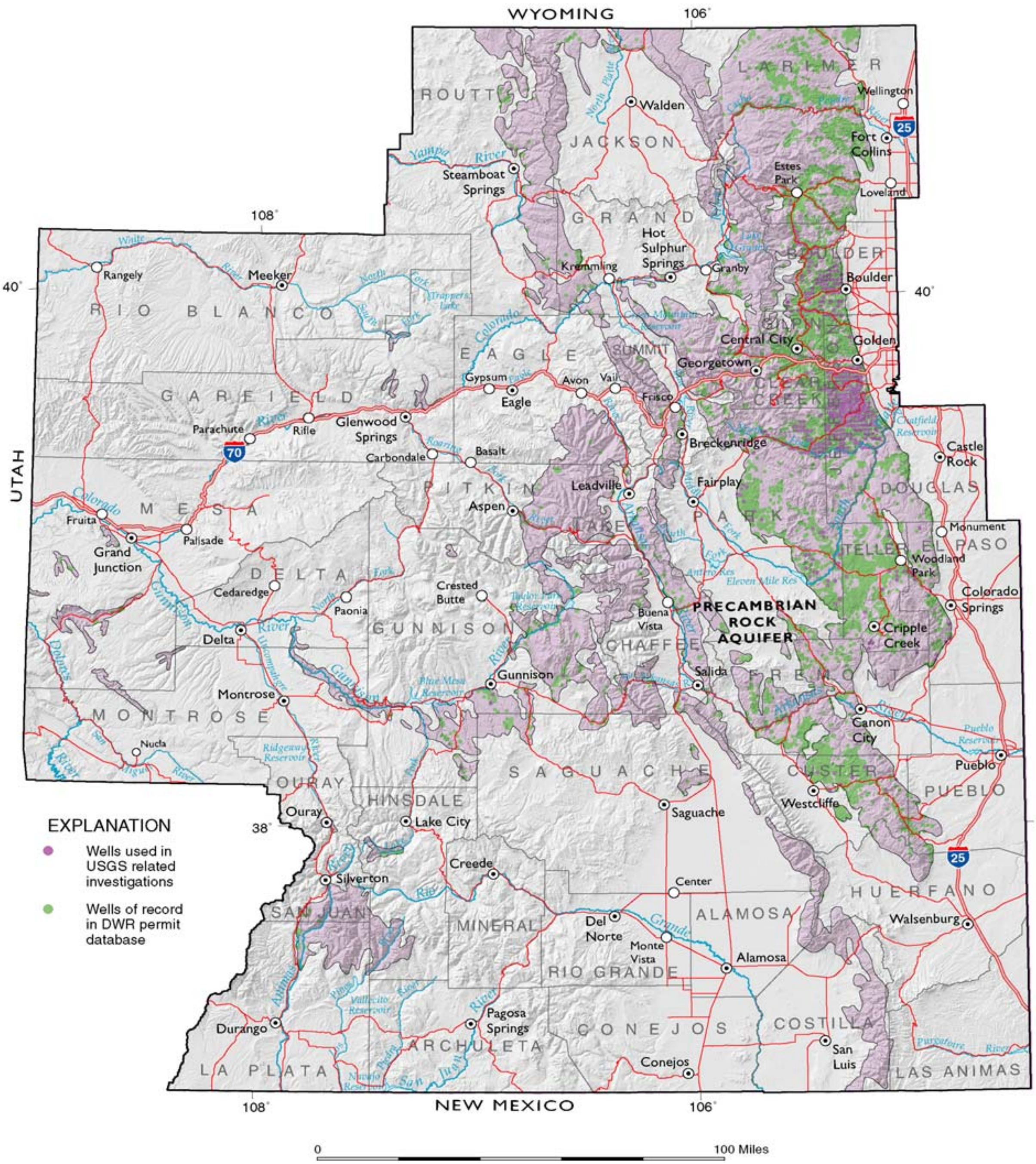

Figure 4. Location and extent of Precambrian crystalline rocks and the distribution of permitted wells therein.

The porosity of Precambrian crystalline rocks is very low — as a general rule, less than 1 percent. Snow (1968 and 1973) reports primary porosity of 0.05 to 0.005 percent and secondary porosity of 0.1 to 0.001 percent. Fractures provide the only significant porosity and flow conduits within the unweathered crystalline rocks of Colorado. Ground-water discharge and storage in crystalline rocks predominantly occurs in fracture networks. Vertical or steeply dipping fractures provide recharge while near-horizontal fractures provide storage capacity and some degree of hydraulic continuity.

Data from the Colorado Division of Water Resources (DWR) well permit database are used to present the distribution, depth, water level, and yield of wells completed in the fractured, crystalline-rock aquifers. Nearly 36,000 water supply wells have been completed within Colorado's Precambrian, fractured, crystalline-rock aquifers. They are distributed throughout the central Rocky Mountain region on those parcels of private land where Precambrian crystalline rocks represent the bedrock (Figure 4). Analysis of well permits of record with the DWR indicates that 90 percent of the wells were completed at depths of less than 550 ft (168 m) (Figure 5). A histogram of the wells representing the 90^{th} percentile of the data indicates a bell-shaped distribution for completion depths with mean and median values of 274 ft (84 m) and 245 ft (75 m) below ground surface, respectively (Figure 6).

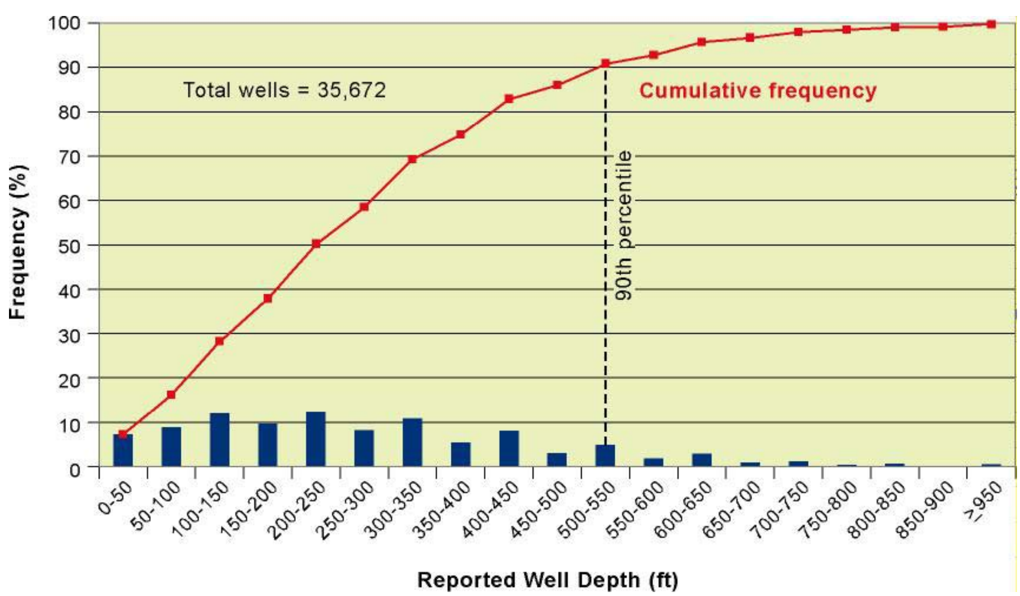

Figure 5. Depth of water wells in the Precambrian crystalline-rock aquifers and their relative frequency of occurrence.

Because overburden pressures tend to close fractures with depth, drillers have historically limited well completions to 300 or 400 ft below ground surface. More recently deeper wells are being drilled, due to increased development pressures. Adequate water supply for domestic purposes has been encountered at depths approaching 1,000 ft.

Depth to water in Precambrian rocks varies depending on topographic position and the amount of fracturing which permits recharge, but generally is within 150 ft or less of the ground surface. The water table in an interconnected system of fractures differs from the water table in porous granular media as the level of saturation is different for each fracture in the system and hydrostatic pore pressure exists only in the fractures and not in the blocks between (Lovelace, 1980). While water levels in neighboring wells may vary because the source is from different fracture zones, the presence of an ambient joint or fine fracture system permits some interconnection between wells.

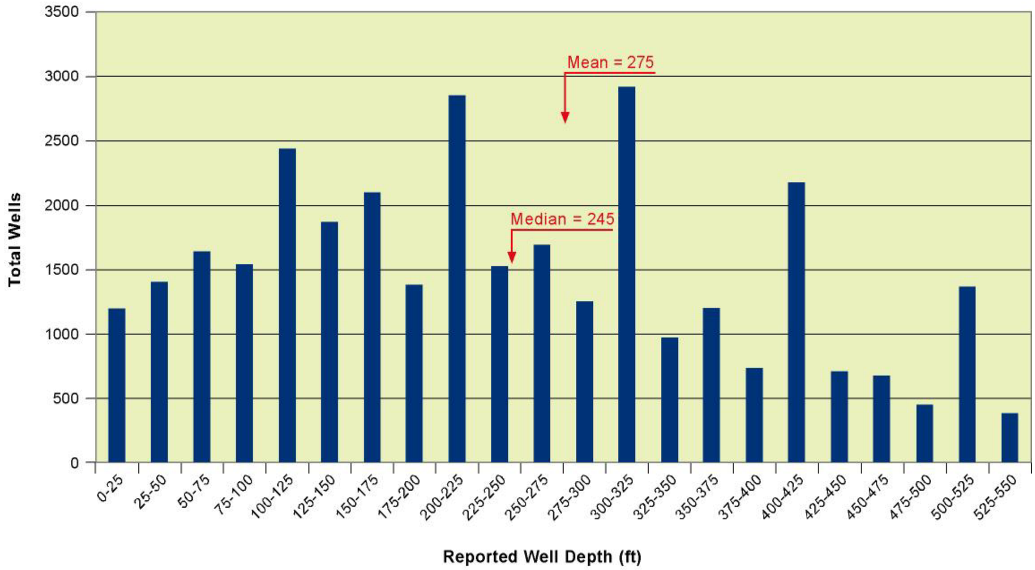

Figure 6. Histogram of well completion depths for the 90th percentile of wells completed in the Precambrian, crystalline-rock aquifer.

The reported depths to water (water level) below ground surface, in the DWR database, vary from the surface (a spring) to the total depth of the well. The distribution of reported water levels by well depth is presented on Figure 7. The water level variability for any given well depth is due in part to the inclusion of all wells completed within Precambrian rocks and the large time period of record. A 30-point moving average trendline has been applied to this data to provide a statistical basis in which to interpret the data. The trendline suggests that reported water levels are generally in the range of 25 to 30 percent of total well depths.

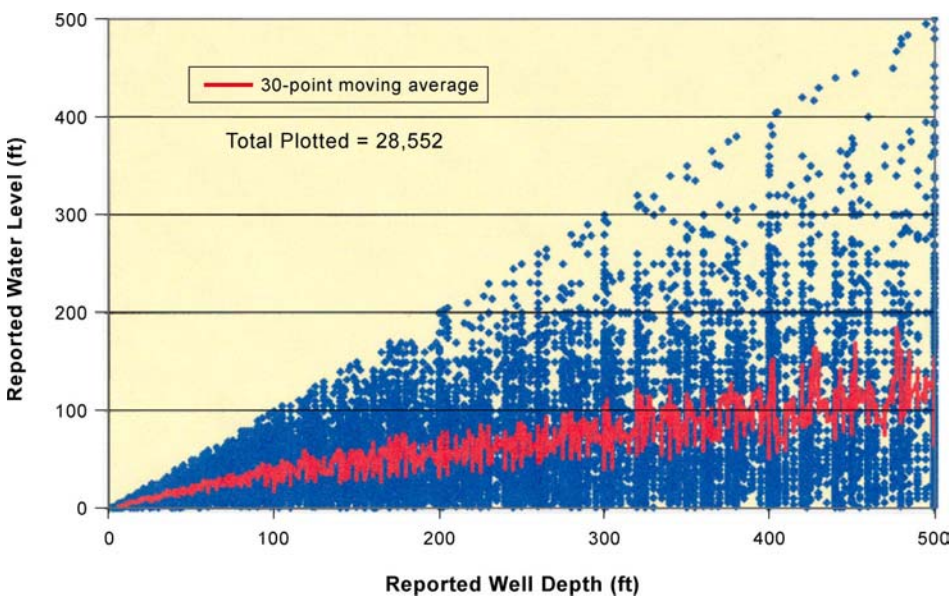

Figure 7. Scatter plot of reported water levels for specific well depths for wells completed in the Precambrian, crystalline-rock aquifer with a moving average trendline.

Well yields from Precambrian rocks are generally only a few gallons per minute (gpm), although wells that penetrate extensively fractured zones, fault zones, or shear zones may produce up to 50 gpm (190 liters per minute, lpm) or more. Analysis of well permit records indicates that 90 percent of the wells completed in Precambrian rocks yield less than 15 gpm (57 lpm). A histogram of reported well yields is presented as Figure 8. There is an inherent bias in these yield values as the state of Colorado limits domestic and stock watering well production to 15 gpm (57 lpm). This stipulation results in a large number of domestic wells reporting a yield of 15 gpm (57 lpm) as documented in the histogram. The well yield data indicate that the majority of these wells produce less than 5 gpm (19 lpm). The mean and median yield values for these data are 7 and 4 gpm (26 & 15 lpm), respectively. The mean yield value reported by the USGS for the Turkey Creek watershed in Jefferson County was 5.6 gpm (21 lpm) (USGS, 2001). Recent investigations (Hicks, 1987; USGS, 2001) suggest that to some extent, well productivity may be related to rock type, amount and orientation of fracturing, and topographic position.

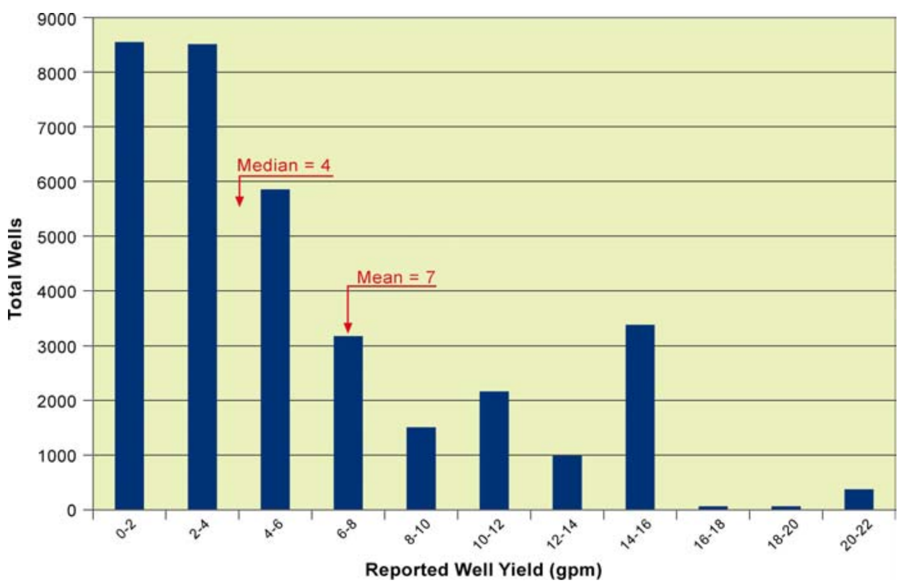

Figure 8. Histogram of reported well yields for the 90th percentile of wells completed in the Precambrian crystalline rocks.

As one might expect from a fractured system, the transmissivity of the aquifer is highly variable. Most of the data available on fractured, crystalline-rock aquifers is from the Front Range area west of Denver. Additional information on the Turkey Creek watershed in Jefferson County can be found in the paper by Poeter and others within this chapter of the publication. Aquifer tests reported by Lawrence (1990) for the Conifer area provide a range of transmissivity values from single digits to 9,300 gallons per day per ft (gpd/ft) (1.15×10^{-6} to 1.34×10^{-3} m^2/s). Investigators working in the Turkey Creek watershed in Jefferson County have reported minimum and maximum transmissivity values of 6 and 12,870 gpd/ft (8.62×10^{-7} to 1.85×10^{-3} m^2/s), respectively (USGS, 2001). In the upper Colorado River Basin, transmissivities in the Precambrian rocks are less than 10 gpd/ft (1.44×10^{-6} m^2/s) (Apodaca et al., 1996).

Hydraulic conductivity values in fractured igneous and metamorphic rocks are reported by Freeze and Cherry (1979) to range from 10^{-2} to 10^{+2} gallons per day per square ft (gpd/ft^2) (5×10^{-15} to 5×10^{-11} m/s). Snow (1968) reports an average for wells in his study of 0.865 gpd/ft^2 (4.08×10^{-7} m/s), with 77 percent of the data falling between 0.05 and 10 gpd/ft^2 (2.3×10^{-8} to 4.7×10^{-6} m/s).

DISTRIBUTION AND HYDROGEOLOGIC CHARACTERISTICS OF TERTIARY IGNEOUS ROCK AQUIFERS

Figure 9 portrays the areal distribution of significant exposures of younger, Tertiary age intrusive and extrusive (volcanic) igneous rocks. The San Juan Mountains contain thousands of feet of volcanic ashes and lava flows that were deposited by widespread volcanic activity 26 to 36 million years ago. A fairly large expanse of volcanic rocks also exists in the Thirty-Nine Mile volcanic field of southeastern Park County and northern Fremont County. Volcanic rocks underlie surficial materials in parts of the San Luis Valley in Conejos and Costilla counties. Volcanic rocks cap the West Elk Mountains in Gunnison County, and this cap is retained in the upstream one-third of the Black Canyon of the Gunnison, where the upper canyon walls have thick sections of breccia and welded tuff (Hansen, 1987). Volcanic rocks are also common in Huerfano County, and Bartlet Mesa in southern Las Animas County has late Tertiary volcanics about 125 ft thick (Zeuss, 1967).

Like their Precambrian counterparts, intrusive igneous rocks of Tertiary age are also subdivided on the basis of composition, such as basaltic and rhyolitic, whereas extrusive rocks are divided by their physical characteristics such as ash flows, lavas, breccia, and tuffs. Volcanic rocks can be classified into general hydrogeologic units depending on the type of rock and method of emplacement. The physical characteristics of volcanic rocks vary greatly. Chemical composition, mineralogy, volatile content, temperature, and mode of extrusion greatly affect their hydraulic characteristics (Wood and Fernandez, 1988). Volcanic materials include tuffs (pyroclastic deposits), breccias, and surface flows. Extrusion features such as flow breccia, clinkers, flowtop rubble, and shrinkage cracks greatly affect the porosity and permeability of volcanic rocks.

Some volcanic rocks, such as ash-flow tuffs and sheet-flow basalts, have very low primary permeability and porosity (Wood and Fernandez, 1988), but fracturing can result in moderate to high secondary porosity. Younger intrusive igneous rocks, primarily granite, have intruded into the Precambrian metamorphic and igneous complexes. These intrusive rocks usually have hydraulic characteristics similar to those of the Precambrian host rocks.

As of February 2001, there were nearly 3,700 wells of record completed in the outcrop areas of Tertiary igneous rocks. The majority of these wells are concentrated near the towns of Del Norte, Westcliffe, Cripple Creek, Glenwood Springs, and Breckenridge (Figure 9). While a few wells have been drilled in excess of 1,000 ft, 90 percent of the wells completed in these rocks are less than 400 ft deep. An analysis of the DWR well permit data indicates that the majority of these wells are less than 200 ft deep with a mean depth of 191 ft and a median depth of 155 ft below ground surface (Figure 10). These data do not distinguish between location and type of volcanic or intrusive rock that may influence the range of well completion depths.

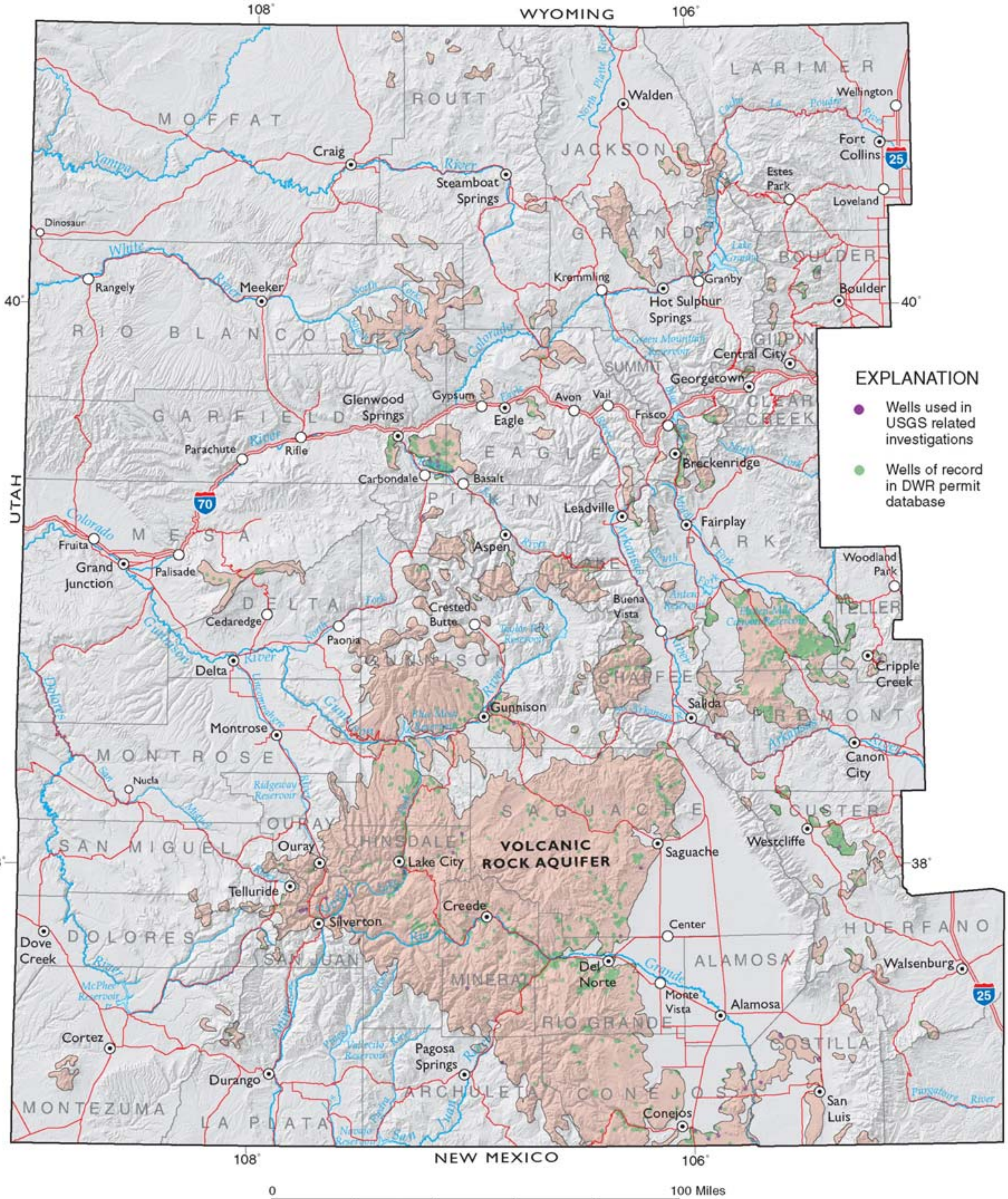

Figure 9. Location and extent of Tertiary igneous rocks and the distribution of permitted wells.

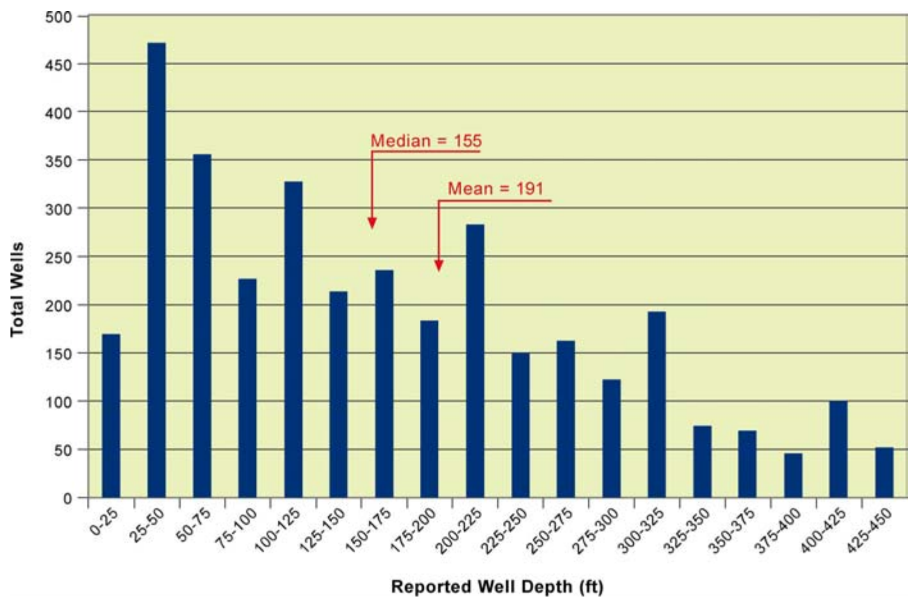

Figure 10. Histogram of well completion depths for the 90th percentile of wells completed in the Tertiary igneous rocks.

Depth to water in the younger igneous rocks also varies considerably depending on location and type of rock. Where these rocks are crystalline, water levels and aquifer characteristics are similar to those in Precambrian crystalline rocks, with water levels generally less than 100 ft deep. In areas where volcanics are interlayered or interfingered with sedimentary rocks, the water levels are generally similar to those in the adjacent sedimentary rocks. Where dikes are present, water levels on opposite sides of the dike may be different due to interference in water flow caused by the dike.

Reported water levels, from the DWR database, for specific well depths are shown on Figure 11. Because these data cover the entire period of record and do not distinguish between location and rock type, reported water levels vary tremendously for specific well depths. A moving average trendline has been applied to provide an estimate of anticipated water levels for various well depths. This trendline indicates that water levels may be anticipated at depths of 40 to 45 percent of the well completion depths common to an area. In general then, water levels in volcanic rock aquifers tend to be deeper than in Precambrian rock aquifers for a given well depth. This may be due to the lack of a regional fracture structure in these younger, relatively undeformed rocks.

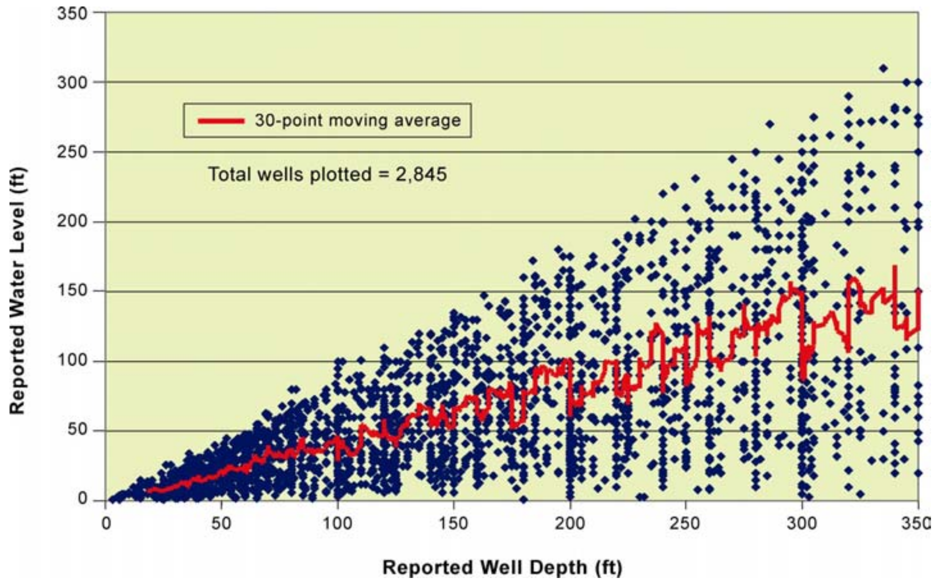

Figure 11. Scatter plot of reported water levels for specific well depths for wells completed in the Tertiary igneous rocks.

As in the Precambrian crystalline rocks, most wells completed in the Tertiary igneous rocks have low yields with 90 percent of the wells of record reporting yields of less than 18 gpm (68 lpm). As discussed previously, the state limits domestic well production to 15 gpm (57 lpm) and permit records are often biased towards this value. Reported well yields in the Tertiary igneous rocks are more evenly distributed, between 2 to 15 gpm (7.6 to 57 lpm), than yields in Precambrian crystalline rocks suggesting that volcanic aquifers are more productive (Figure 12). The mean yield from these data is 14 gpm (53 lpm) and the median value is 10 gpm (38 lpm).

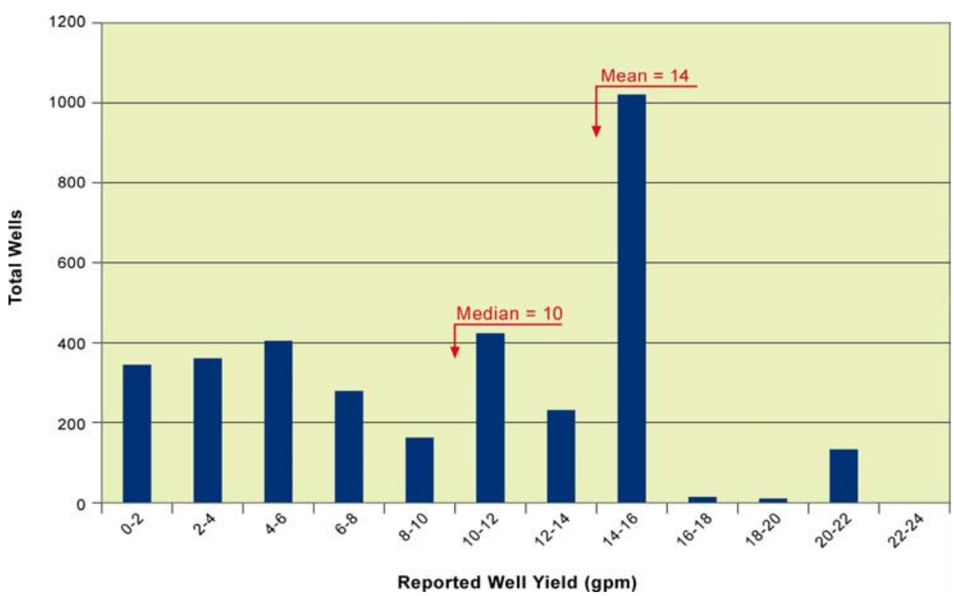

Figure 12. Histogram of reported well yields for the 90th percentile of wells completed in the Tertiary igneous rocks.

In volcanic rocks, water is located in and moves through open voids. Hydraulic properties of volcanic aquifers vary considerably due largely to the variation in rock type and the way the rock was ejected and deposited. Porosity, hydraulic conductivity, and transmissivity are extremely variable due to the localization of specific types of voids. Hydraulic conductivity values for various volcanic rock types are presented in Figure 13.

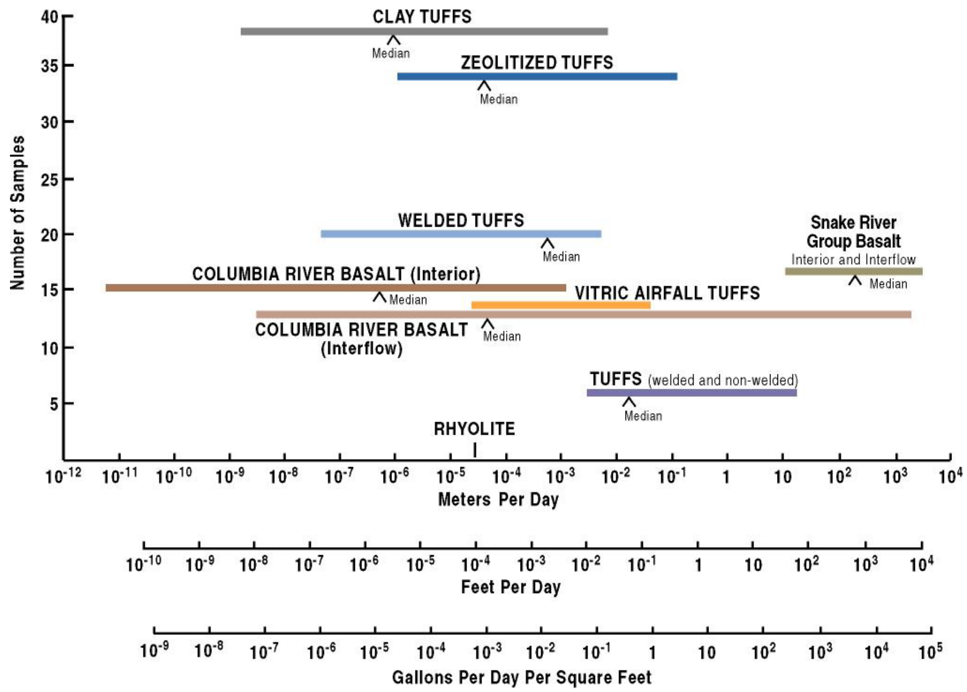

Figure 13. Range in hydraulic conductivity values of volcanic rocks. (from Wood & Fernandez, 1988)

GROUND-WATER USE AND WITHDRAWALS

Population growth in the mountainous regions of Colorado, over the past decade, has placed tremendous demands upon the fractured, crystalline-rock aquifers. Over 30,000 small-diameter wells have been drilled to accommodate domestic and some limited public water supply. In addition to supplying individual residences, these wells supply campgrounds, trailer parks, and smaller subdivisions. Where yields are sufficient, limited municipal supplies have also been developed. Because of the low porosity of these aquifers, they are incapable of storing or transmitting large quantities of water. While no data are publicly available for specific ground-water withdrawals from the crystalline-rock aquifers as a whole, the USGS has estimated 1995 ground-water withdrawals by county (Solley, et al., 1998). In general, these data indicate that countywide annual ground-water withdrawals, in the counties dominated by crystalline rocks, do not exceed 1,000 acre-ft (1.23×10^6 m^3) per year. These low ground-water withdrawal values are more reflective of the land use, availability of surface water resources, and low population densities versus amount of water in storage. Larger developments proposed in the mountainous regions of Colorado must, however, consider the limitations of these aquifers in their decision making process if surface water resources are not available.

In Colorado, volcanic rocks are also not a major source of water. In part, this is because the rocks are not highly productive, but it is also a function of low population densities as public lands dominate in the areas of volcanic rocks. Ground-water withdrawals from volcanic rocks are primarily used for domestic purposes. In some areas of Colorado, the Tertiary volcanic and intrusive aquifers provide geothermal resources.

SUMMARY

Colorado's crystalline rocks consist of Precambrian age igneous and metamorphic rocks and Tertiary age igneous rocks. These rock types are exposed in the mountainous portions of the state, and generally lie within the Southern Rocky Mountains physiographic province. Crystalline rocks occupy approximately 19 percent of the state's total area, and represent the fractured-rock aquifers that supply much of the domestic water supply needs in the mountainous portions of Colorado.

Ground-water discharge and storage in crystalline rocks predominantly occurs in fracture networks. The intensity and width of fractures and the degree of connection between fractures influence the hydraulic characteristics of these aquifers. The aquifer system is unconfined with water levels fluctuating seasonally. The configuration of the water table tends to mimic the surface topography.

A thin veneer of surficial deposits, generally less than 8 ft (2.4 m) thick, typically overlies the fractured, crystalline bedrock. The material in this surficial layer retains soil moisture to recharge the underlying aquifer. Recharge to the fractured, crystalline-rock aquifers is predominantly by infiltration of precipitation and snowmelt, with the majority of recharge occurring between the middle of May and the first part of July. Recharge also occurs where fractured rock underlies saturated alluvium, water storage reservoirs, or individual septic disposal systems. Due to high evapotranspiration rates, only 10-15 percent of total precipitation enters the ground-water storage system, which includes the surficial soil layer. As such, this aquifer can be easily overexploited.

Precambrian crystalline rocks occupy about 12 percent of the surface area of Colorado. These rocks are typically subdivided on the basis of mineral composition. The fracture porosity of Precambrian crystalline rocks is less than 1 percent. Nearly 36,000 water supply wells of record have been completed within this aquifer. Reported well completion depths indicate that 90 percent of these wells are completed at depths less than 550 ft (168 m), with mean and median values of 274 (84 m) and 245 ft (75 m) below ground surface, respectively. Depth to water in Precambrian rocks varies depending on topographic position and the amount of fracturing, but generally it is within 150 ft or less of the ground surface. Due to the low fracture porosities well yields from Precambrian rock aquifers are also low, with mean and median values of 7 and 4 gpm (26 & 15 lpm), respectively.

The younger, Tertiary age intrusive and extrusive (volcanic) igneous rocks generally lie west of and between the outcrops of Precambrian rocks. Like their Precambrian counterparts, intrusive igneous rocks of Tertiary age are also subdivided on the basis of composition, whereas extrusive rocks are divided by their physical characteristics. Chemical composition, mineralogy, volatile content, temperature, and mode of extrusion greatly affect the hydraulic characteristics of volcanic rocks. As of February 2001, there were nearly 3,700 wells of record completed in Tertiary igneous rocks. Well completion depths tend to be shallower than in the Precambrian crystalline rocks with the majority of wells less than 200 ft deep. Water levels within the Tertiary igneous rock aquifers tend to be within 100 ft of ground surface. Reported well yields tend to be slightly higher than in the Precambrian crystalline aquifers with a mean yield of 14 gpm (53 lpm) and the median yield of 10 gpm (38 lpm).

Population density in the mountainous regions of Colorado is generally fewer than 39 residents per square mi (2.59 sq. km). In the past ten years, Colorado's population within the central mountain regions has grown at nearly 42 percent. This significant increase in population has placed tremendous demands upon Colorado's fractured, crystalline-rock aquifers. The resource pressure tends to be localized as land use is primarily public national forest. Tens of thousands of small-diameter wells have been drilled in the mountainous regions of Colorado to accommodate domestic and some limited public water supply. Because of the low fracture porosity of these aquifers, they are incapable of storing or transmitting large quantities of water.

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CASE HISTORY OF AQUIFER STORAGE AND RECOVERY IN THE DENVER BASIN AQUIFERS

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Key terms: aquifer storage, Arapahoe aquifer, conjunctive use, Denver Basin, ground water, injection, recharge, thermal gradients

ABSTRACT

The Denver Basin Aquifer Recharge Demonstration Project (Demonstration Project) was conducted as part of the High Plains States Groundwater Demonstration Program under the auspices of the U.S. Department of the Interior, Bureau of Reclamation. The Demonstration Project provided 6 years of research into aquifer storage and recovery in the Denver Basin aquifers and its implications for full-scale conjunctive use projects in the Denver Basin as it related to hydraulics, water quality, economics, and in-situ issues. The project was initiated in 1990 and completed in 1997.

The principal hydraulic effect associated with injection that was observed during the course of the Demonstration Project was related to the large temperature differential between the water being injected and the ambient Arapahoe aquifer temperature. Detrimental shifts in water quality were not observed throughout the course of the Demonstration Project, other than the native water quality generally shifting from a calcium-sulfate water to a predominantly calcium-bicarbonate water, which closely resembled the initial quality of the Denver source water. The economic feasibility analysis estimated a total cost to buy the water, deliver the water to an injection well, inject into the Arapahoe aquifer, store in the aquifer, and then recover the water for subsequent use is \$2.45 per 1000 gallons (\$0.65 per m^3). During the Demonstration Project, rules were promulgated related to artificially recharging water into the Denver Basin aquifers. The Denver Basin Artificial Recharge Extraction Rules were promulgated, effective July 1, 1995.

INTRODUCTION

Most water suppliers along the Front Range of Colorado, outside the City and County of Denver and the City of Aurora, depend heavily on the water resources of the Denver Basin aquifers to meet municipal water supply demands. The Denver Basin underlies the metropolitan areas of the Front Range from Greeley to Colorado Springs (Figure 1). Containing over 250,000,000 ac-ft (1×10^{12} m^3) of recoverable water (Robson, 1984), the Denver Basin aquifers would seem to provide a virtually inexhaustible supply of water, given a current annual demand of approximately 60,000 ac-ft (2.43×10^8 m^3). However, only a small portion of the Denver Basin aquifers have been developed in the Denver metropolitan area, with large portions of the basin being virtually undeveloped. While there is a large volume of water in storage, because of the concentrated area of development water providers are currently experiencing 20 to 30 ft (6.1-9.1 m) of water level declines per year, which results in declining well production.

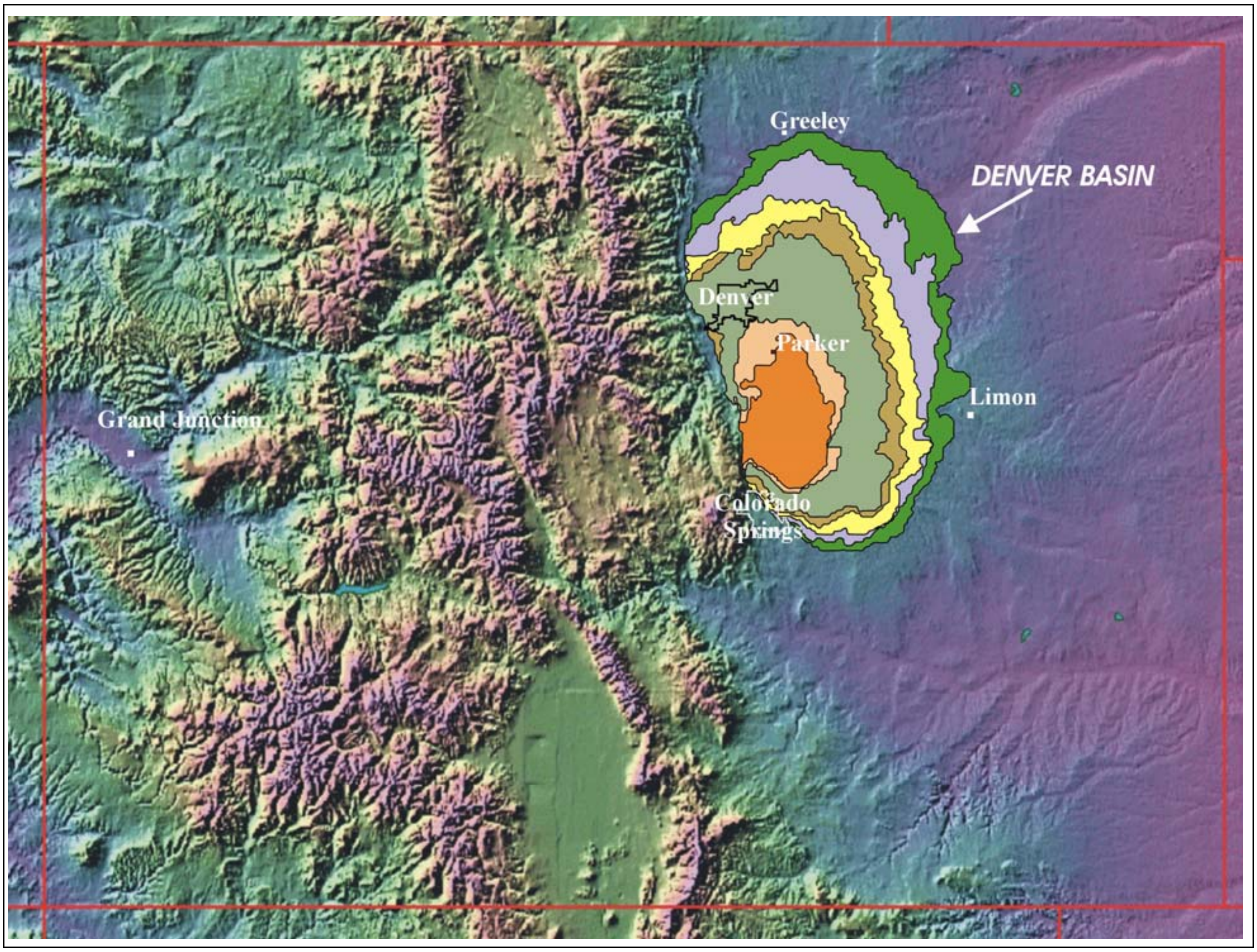

Figure 1. Location Map of Denver Basin in Colorado

Because water is being extracted from the Denver Basin at a rate greater than it is being recharged, the aquifer system is currently being mined. One water supply option available to help mitigate the effects of mining water from the Denver Basin aquifers is the conjunctive use of surface water supplies with existing ground water supply systems.

To evaluate the feasibility of a conjunctive use system, i.e., the feasibility of aquifer storage and recovery using non-native surface water supplies as the source water for injection, a 6-year research project was initiated under the auspices of the United States Bureau of Reclamation's High Plains States Groundwater Demonstration Program. A large ground water-dependent water supplier in the southeastern Denver metropolitan area (Willows Water District) used its existing Denver Basin aquifer well system to evaluate if surface water supplies could be injected into the Denver Basin aquifers (specifically the Arapahoe aquifer), stored in the aquifer for a period of time, then recovered at a later date to meet demands. The surface water supplies for the research project were provided by Denver Water through its existing system. This supply essentially consisted of snowmelt runoff water collected in high mountain reservoirs, routed to Denver, treated, and provided to the Willows well through Denver Water's potable water distribution system.

There are a number of reasons why a conjunctive use plan was evaluated, which include (a) declining water levels in the Arapahoe aquifer indicated a finite water supply source, (b) excess surface water supplies in the South Platte River were potentially available during wet years, (c) conjunctive use of surface water and ground water supplies would extend the life of the Arapahoe aquifer, (d) increased beneficial use of scarce water supplies would be a prudent water supply management tool, (e) optimization of existing surface water storage, surface water treatment, transmission facilities and ground water wells would provide better economic benefits for the capital costs of installation, (f) confined aquifer conditions could be used to provide transmission to remote locations to efficiently apply the conjunctive use process.

The conjunctive use concept was identified as a viable means to take existing injection technology and use it in a unique way to optimize beneficial use of available surface water by utilizing underground aquifers as an environmentally-sound storage vessel. The technical, legal and institutional issues related to a full-scale injection, storage and recovery plan were developed as part of this research.

The technical aspects of the research involved a cyclical process of injection runs, with intervening pumping cycles, to evaluate the hydraulic and water quality effects on the Arapahoe aquifer as a result of the injection process. The legal and institutional issues involved promulgating rules and regulations that would allow non-native Denver Basin aquifer water to be injected and stored in the Denver Basin aquifers and to provide a mechanism for the subsequent recovery of this water within the Colorado Water Rights Law system. The relative cost of an injection, storage and recovery project was compared to the direct use of either surface water or ground water supplies to assess economic viability.

SITE CONDITIONS

Arapahoe Aquifer Hydrogeology

The Arapahoe aquifer, the target aquifer for the research, is part of the Denver Basin, which covers approximately 6700 mi^2 (17,367 km^2) along the Front Range of Colorado (Figure 1). The principal water-bearing units of the Denver Basin, in stratigraphic succession from the lowest to the highest, include the Late Cretaceous Laramie-Fox Hills sandstone, the Late Cretaceous Arapahoe Formation, the Late Cretaceous and Early Tertiary Denver Formation, and the Tertiary Dawson Formation, The non-water bearing Late Cretaceous Laramie Formation separates the Arapahoe Formation from the Laramie-Fox Hills sandstone (Figure 2).

The Arapahoe aquifer, contained within the Arapahoe Formation, generally consists of a 400- to 700-ft (121-213 m) thick sequence of interbedded sandstones, siltstones and shale. The Arapahoe aquifer typically may have seven to ten distinct sandstone units that are capable of producing water to wells. However, for purposes of water rights determinations, the State Engineer's Office (SEO) considers these interbedded sandstone units as a single hydrologic unit, and wells are typically screened across all the sandstone units rather than individual wells for each sandstone unit within the Arapahoe aquifer.

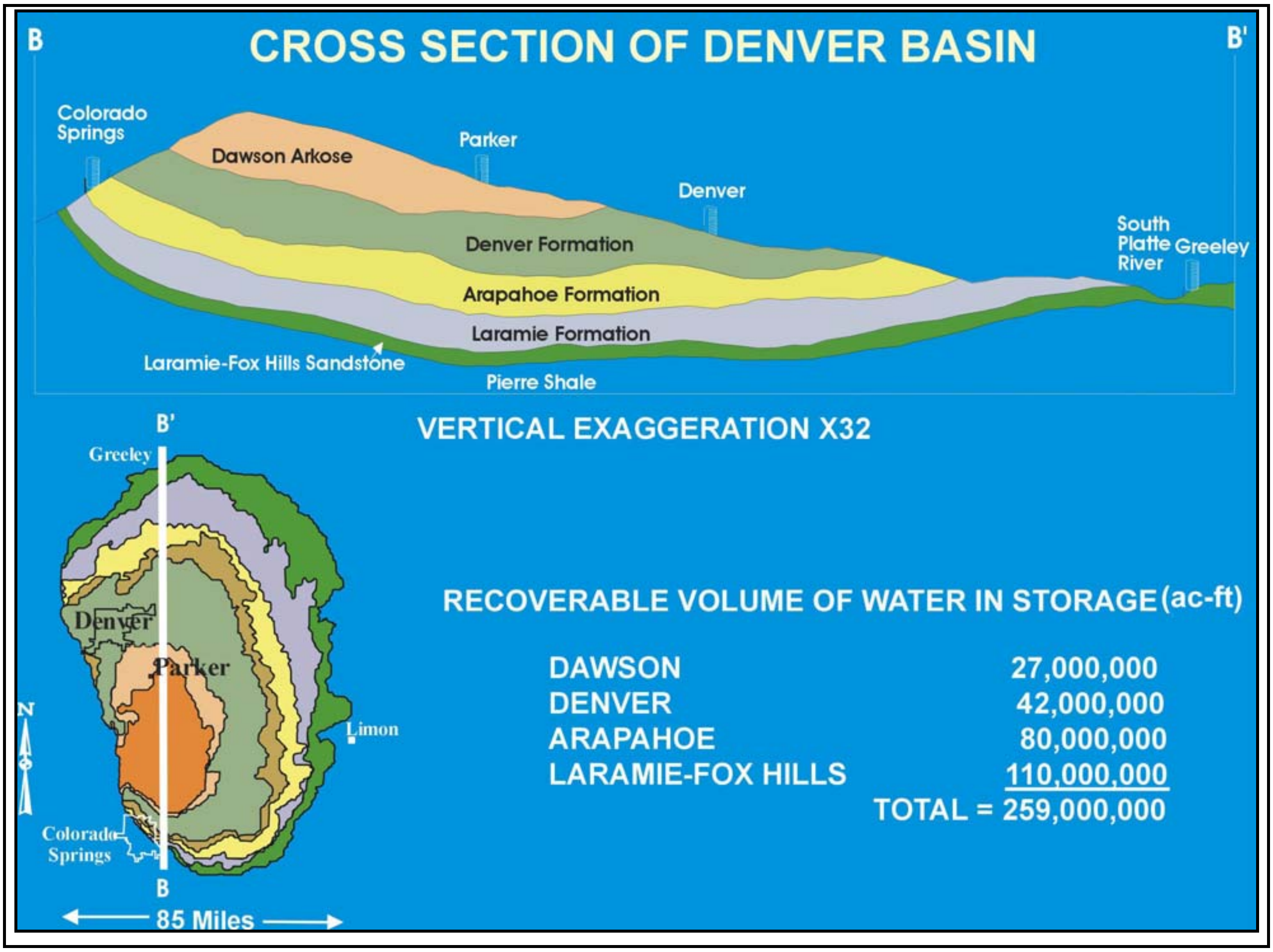

Figure 2. Cross-Section of the Denver Basin

In the deep, central portion of the Basin, the upper sandstone units of the Arapahoe aquifer are fine-grained and angular, while the lower Arapahoe sandstone units tend to be more coarse-grained and rounded in nature. The thickness of each of these sandstone units tends to increase with depth in most areas and typically have thick sequences of sand in the basal portion of the aquifer.

At the research well, designated as A-6A, the Arapahoe aquifer saturated thickness is approximately 310 ft (94.5 m) and the hydraulic conductivity was 8.5 ft/day (2.6 m/day). The aquifer transmissivity was 19,800 gpd/ft (2.8×10^{-3} m^2/sec), and the well is built with 20-in (50.8 cm) outside diameter casing, wire-wrapped stainless steel screen and engineered gravel pack (Figure 3).

In the 1800s, wells completed in the Arapahoe aquifer had artesian flow, i.e. water flowed at the surface in some areas of the basin. In the central portion of the Denver Basin, current water levels in the Arapahoe aquifer are 500-600 ft (152.4-182.9 m) below ground surface, and Arapahoe aquifer water levels are declining at an annual rate of approximately 20-30 ft (6.1-9.1

m). With the increase in use of water from the Arapahoe aquifer, it is projected that this trend will continue, if not increase, as long as the aquifer remains confined, i.e. water levels are below ground surface, but above the top of the aquifer.

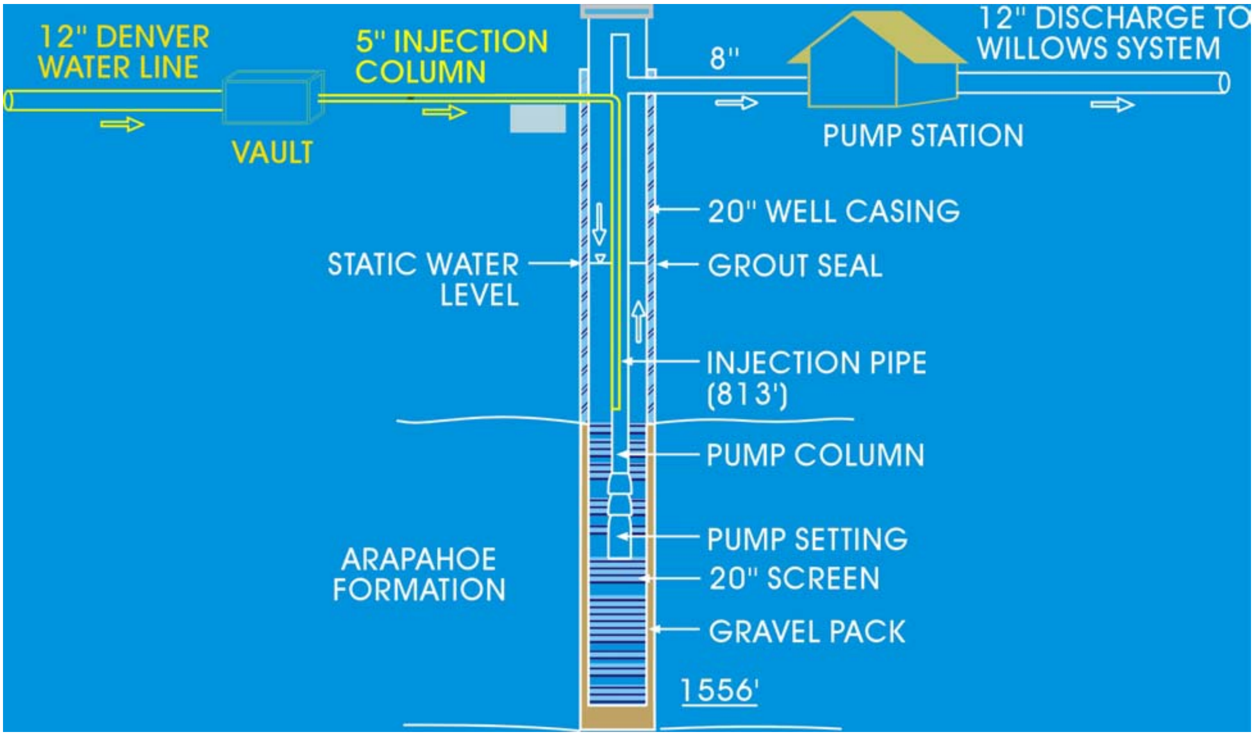

Figure 3. Production Well Completion and Injection Retrofit

While water levels have declined significantly over time, and will continue to do so, the Arapahoe aquifer over much of the Denver Basin still is in a confined condition, i.e. the water in the Arapahoe aquifer is confined under hydrostatic head greater than atmospheric pressure by overlying relatively impermeable strata. When a well under confined conditions is pumped, the resultant movement of water to the well is a function of the elastic properties of the aquifer. Under confined conditions, hydrostatic head is reduced due to pumping, which results in an increased aquifer load. This increase in the aquifer load results in a compression of the aquifer that forces some water from it. Additionally, this lowering of the hydrostatic head causes a small expansion of the aquifer, with a subsequent release of water (Todd, 1980). Simply stated, assuming a non-leaky aquifer, producing water under confined conditions results in a lowering of head in the <u>entire</u> confined portion of the aquifer. Likewise, injection of water into a confined aquifer simply increases head in the <u>entire</u> confined portion of the aquifer.

This phenomenon that occurs under confined aquifer conditions means that if water is injected at one point in the Arapahoe aquifer, there is a resultant head increase across the entire confined portion of the aquifer. Therefore, if water is extracted from a remote location in an amount equal to that injected, the overall head in the confined portion of the aquifer will simply be reduced to the original condition prior to injection. This is one of the key components of injection and extraction in the Arapahoe aquifer of the Denver Basin.

Pumping water from a well under confined conditions differs from pumping a well under unconfined conditions. In unconfined conditions, the water produced from the well is drained from the sand in the vicinity of the well, and pumping and/or injection of water produces a much more localized effect than pumping and/or injection under confined conditions. This phenomenon is important to understand, as it plays a significant role in how injected water can be extracted under unconfined conditions versus confined conditions under the rules and regulations promulgated by the SEO.

The Arapahoe aquifer in particular, and the Denver Basin in general, has a large storage potential, based on current water level data. It is estimated that the Arapahoe aquifer has potentially 140,000 ac-ft (5.67 x 10^8 m^3) of storage and the Denver Basin (including all four aquifers) has a potential storage capacity of 500,000 ac-ft (2.0 x 10^9 m^3). This estimate is based on an average water level across the basin, the area in each aquifer, and a storage coefficient of $1x10^{-4}$. By comparison, the largest reservoir in Colorado, the Blue Mesa Reservoir, has a storage capacity of approximately 800,000 ac-ft, and Lake Dillon has a storage capacity of 254,000 ac-ft.

INJECTION SETUP

Downhole Injection Retrofit

To inject water into an existing well, an injection column had to be installed in conjunction with the existing pump column and pump. In this way, water could be alternately injected and pumped out, not only to keep the well clean during the injection process, but also to allow the injected water to be subsequently pumped to use. To minimize air entrainment in the well that could damage the well, the pump, and the aquifer, the injection column was designed to always maintain positive pressure at the surface.

The first phase of the project was to bring the water to be injected to the injection well. This was accomplished by the installation of a pipeline. Once the source water was brought to the site by pipeline, the second phase was to construct the well retrofit to allow the water to be injected down the borehole. Based on the inside diameter of the A-6A casing (approximately 19 1/3 in) (49 cm), the diameter of the existing column pipe to the A-6A pump (8 in) (20.3 cm) and the expected range of injection rates (400 to 1,200 gpm) (25.2-75.8 L/sec), a 5-in. (12.7 cm) inside diameter (I.D.) injection pipe was installed alongside the existing pumping column pipe. A schematic of the downhole injection retrofit is shown in Figure 3.

To accommodate the high range of expected injection flow rates and to maintain a positive pressure within the injection column, an orifice assembly was designed and installed at the downstream end of the injection pipe. The orifice assembly was also designed to be changed out as the study progressed by the use of a wire line (without requiring the injection pipe to be removed from the well). Changes in the orifice assembly that increased the orifice opening allowed the injection flow rates to be increased without providing surface booster pumping. The use of the orifice assembly also maintained the injection piping with back pressure at the surface.

Monitoring Well Installation

To supplement data collection efforts at the injection well (A-6A), a well was installed approximately 110 ft (33.5 m) northwest of A-6A to monitor water level and water quality data. Monitoring well M-1 was constructed using 6-3/8 in. (16.2 cm) I.D. steel casing and wire-wrapped screen. Screened intervals in M-1 coincided with the same screened intervals in A-6A, so that both wells are completed in the same production interval.

PROJECT OPERATIONS

Methodology of Recharge/Pumping Cycles

The primary objective of the project was to define the engineering issues associated with injecting treated surface water into, and recovering this water from, Denver Basin bedrock wells. To meet this objective, several criteria were established and evaluated to help demonstrate that short-term injection rates can be maintained over the long term. These criteria were designed to assess whether any long-term degradation was occurring to the well, or to the aquifer, as a result of the injection process.

The principal criteria that were monitored included (a) injection flow rates, (b) injection run durations and the associated duration of the pump cycles, (c) water level changes in the well due to injection and pumping, with related evaluation of aquifer and well hydraulic characteristics, (d) back pressure at the well head, (e) sand production at the well due to injection, and (f) water quality monitoring of both the source water at the well head and the water pumped from the injection well and the monitoring well.

During the project, water was injected for predetermined periods, ranging from one to six weeks. During the early injection runs, the injection rates were kept constant for the duration of the test to allow comparison of the actual head buildup during injection to the theoretically-expected buildup. By varying only injection duration or the injection rate during the course of each test, the relative effects of both variables (duration and changing rates from one run to another) on the well and the aquifer could be assessed over the course of the multiple test runs.

However, when higher injection rates were used during the latter stages of the project, flows could not be maintained at a constant rate because there was insufficient surface pressure to drive the higher rates as the head built up. Therefore, during these latter runs, the head buildup in the well, the system pressure and the orifice opening dictated the injection rate, with rates starting out high, then tapering off later in the run.

Following an injection run, the water levels in the injection well were allowed to stabilize, usually for a period of 48 hours, before a pump cycle was initiated to clean the well. During the pump cycle, the well was pumped at a constant rate in the range of 1,000 gpm (63.1 L/sec). Water levels in Wells A-6A and M-1 were recorded during the pump cycles for analysis of well and aquifer characteristics. Sand production was also monitored closely during the pumping cycles using a Rossum sand tester. Sand production was monitored to evaluate the possibility of any destabilizing effects on the bore hole face.

Water samples were collected weekly from the Denver source water during an injection run, and from Well M-1 and A-6A during the pump cycle between each injection run. During the injection runs, several field water quality parameters were continuously monitored using a flow through cell and data logger, to assess changes in the injection source water quality. These field water quality parameters included pH, Eh, specific conductance, temperature, dissolved oxygen, and free and total residual chlorine. Back pressure at the well head was also monitored during the injection runs so that positive pressure was maintained in the injection pipe at all times.

DISCUSSION OF PROJECT RESULTS

The principal focus of the project was to evaluate the hydraulic and water quality effects related to deep well injection, storage and recovery. In addition, technical feasibility of the conjunctive use program has to be accompanied by an assessment of its economic viability and it needs to meet both institutional and legal requirements to make it viable for implementation on a full-scale basis.

Hydraulics

Thirty-four injection runs and pump cycles were completed as part of the 6-year project. During the course of the project the two principal effects observed that were related to the hydraulics of the injection process were (a) head buildup in the well during the injection process greater than what would theoretically be expected and (b) a progression of increasing drawdown in the well during the pump cycles as the project progressed.

Theoretically, head buildup in the injection well for a given injection rate should be a mirror image to the drawdown observed from a well pumping at the same rate. However, throughout the project, the head buildup in the injection well was significantly greater than what would theoretically be expected. Figure 4 shows the comparison of the theoretical A-6A head buildup response compared to the actual response from an early injection run at 450 gpm (28.4 L/sec). This figure shows that, not only was there approximately 250 ft (76.2 m) of additional head buildup above what would theoretically be expected, but the slope of the head buildup line was also steeper than the theoretical line, as shown in Figure 4. Also of note is that the actual head buildup response at monitoring well M-1 parallels the theoretical response, indicating that water level buildup in the Arapahoe aquifer remote from the injection well follows the theoretical response. This observation also indicates that the variation from the theoretical response at A-6A is a near-borehole phenomenon.

Effects of Temperature Differentials – The data collected during the project indicate that both hydraulic phenomena of excess head buildup during injection and increasing drawdowns during pumping are related to the temperature differential between the ambient aquifer temperature and the temperature of the water being injected. The temperature and fluid resistivity survey indicated that the temperature in the Arapahoe aquifer increases with depth, with a temperature

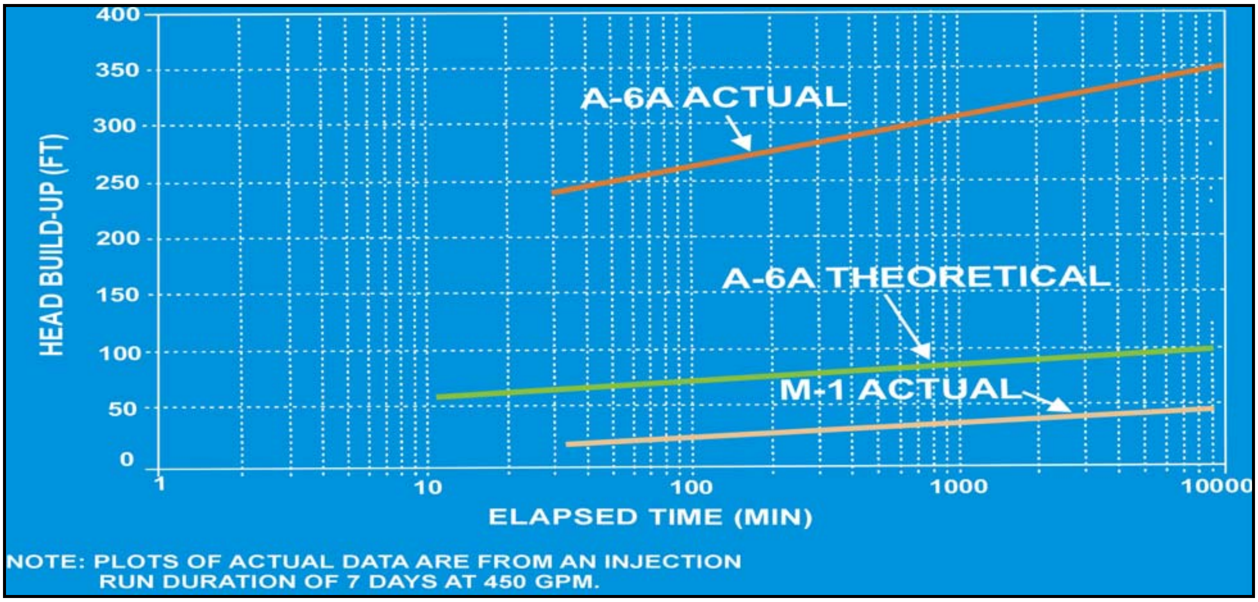

Figure 4. Typical Head Buildup Plots

of approximately 72.5 °F at 1000 ft. (22.5 °C at 304.9 m), grading to a temperature of 79.7 °F at 1500 ft. (26.5 °C at 457.3m) (JCHA, September 1992). When a 24-hour constant-discharge pump test was conducted at well A-6A prior to any injection, the composite temperature from the water being pumped was 71.2 °F (21.8 °C). Conversely, at the start of injection run IR-1, the average source water temperature was 39.6 °F (4.2 °C) and the injected temperature during the course of the project ranged from 34.9 to 58.6 °F (1.6 to 14.8 °C).

The large differences in temperature between the source water being injected into the Arapahoe aquifer and the ambient temperature of native Arapahoe aquifer water led to a significant change in the dynamic viscosity of the water. This is illustrated in Figure 5, which shows the range of temperature variation between the ambient aquifer temperature and the injected temperature and the resultant range of dynamic viscosity. As Figure 5 shows, as the temperature changes from the ambient aquifer temperature to the lowest temperature of the source water injected (34.9 °F or 1.6 °C), the dynamic viscosity increases by 70 percent.

The effect that this will have on the hydraulics of the injection process, and the subsequent pumping of the injection well, can be shown by the relationship of aquifer hydraulic conductivity to the dynamic viscosity. Hydraulic conductivity (K) can be expressed by the following equation:

$$K = k\rho g / \mu$$

where, k= intrinsic permeability
ρ = fluid density
g = acceleration due to gravity
μ = dynamic viscosity

Given that k is constant for a given bedrock type, g is constant, and ρ changes only to a minor degree with temperature changes, as the dynamic viscosity (μ) increases the aquifer hydraulic

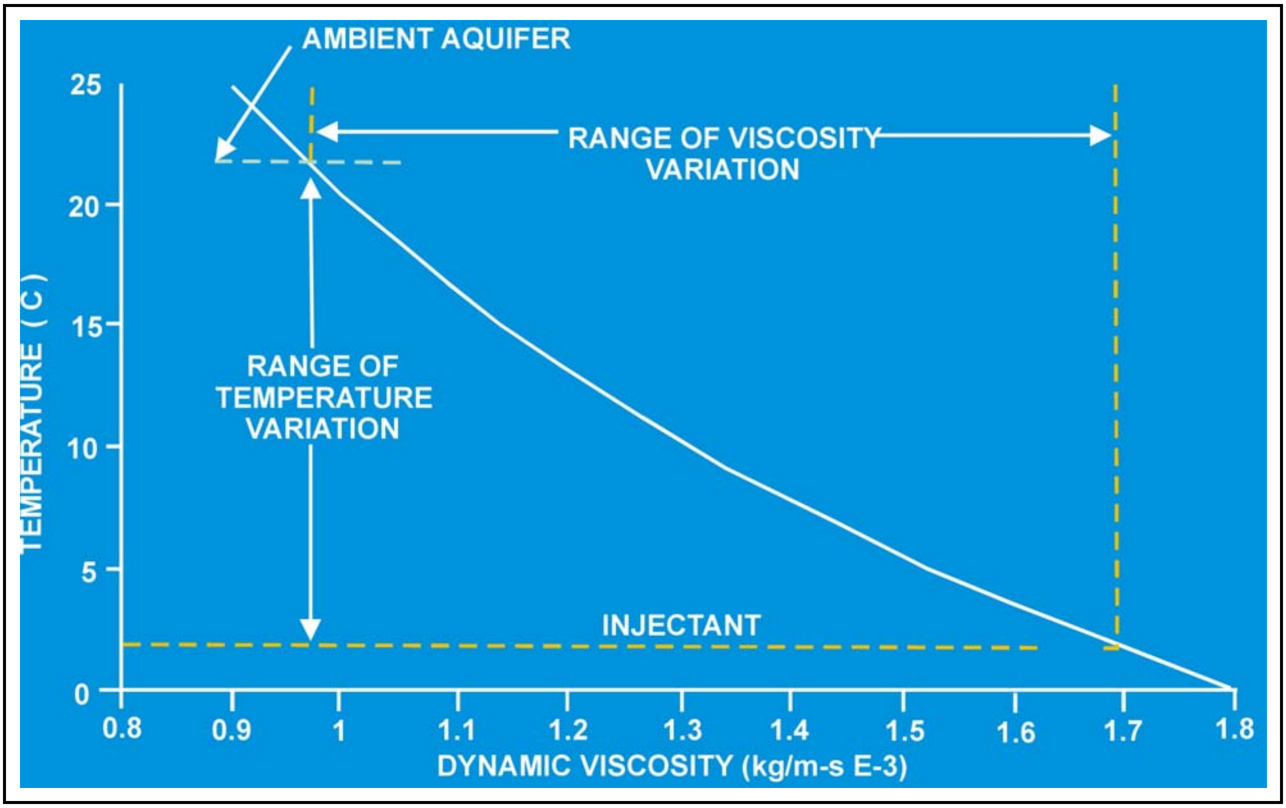

Figure 5. Effect of Temperature Change on Dynamic Viscosity

conductivity (K) decreases linearly. This decrease in hydraulic conductivity then produces a non-linear change in flow rate. Therefore, as cold water is injected into a warmer aquifer the viscosity change caused by this cold water mass will produce a corresponding decrease in the aquifer hydraulic conductivity. When the change in hydraulic conductivity is superimposed on the head buildup plots shown in Figure 4, there is a very good match. It is believed that it is this phenomenon that creates the head buildup in the well that is greater than should theoretically be expected and also creates the additional drawdown in the well during the pump cycles. This is likely related to the non-uniform mixing of different temperature waters.

The result of increasing drawdown during the pump cycles is a reduction in the well production rate and, therefore, additional pumping costs to pump an equal volume of water from a well. While temperature effects need to be considered as an operational constraint, it does not appear that temperature effects are a fatal flaw, which would preclude injection of colder surface water supplies into a warmer aquifer matrix.

Effects of Other Factors – While the data indicate that water temperature is the principal factor that affects the observed increased head buildup during injection and an increased drawdown during the pump cycles, there were a number of other factors that were evaluated as part of the project which are common problems encountered in injection well processes. These processes include:

(1) Suspended sediment in the recharge water causing clogging of the screen and/or gravel pack.

(2) Entrained air in the recharge water, which can result in two-phase flow if the air is forced out into the formation. This ultimately can cause air locking of the formation.

(3) Microbial growth in the well that results in slime buildup which can plug the screen and/or gravel pack surrounding the well.

(4) Chemical reactions between the source water and the native Arapahoe aquifer water, which could cause precipitation that can clog the screened interval and/or gravel pack.

(5) Chemical reactions between the source water and the formation matrix that could result in dispersion of clay particles that could reduce the permeability in the vicinity of the injection well.

(6) Geochemical reactions that could occur in the Arapahoe aquifer, as it is a naturally reducing environment and the recharge water is oxidizing in nature. Therefore, there is the potential for iron and manganese precipitation due to the change in the redox potential in the vicinity of the well.

While data were collected throughout the project to address these other potential factors that could negatively impact the injection process, none of the data collected indicated that any of these factors were significantly contributing to the hydraulic effects described above.

Suspended sediment was never a problem as part of the Demonstration Project, as all total suspended sediment (TSS) analyses of the Denver Source water have indicated an extremely clear, sediment-free water.

It appears that the only suspended particles that have been introduced to A-6A are related to scouring from either the casing as a result of high velocities exiting the injection pipe or from the distribution pipe. Iron flakes were observed during the initial stages of each pump cycle, which were judged to be related to the casing. However, it does not appear that any of these materials were causing significant clogging of the screen and/or gravel pack, as these materials were periodically removed by the pump cycles.

Entrained air was never introduced to the Arapahoe aquifer, other than some potentially small amounts during the initial filling of the injection column (usually the first few minutes of each injection run). However, no entrained air was ever observed during any of the pump cycles. It is not believed that this was a factor in the observed reduction in well specific capacity.

Iron-related bacteria (IRB) were identified in well A-6A and it was postulated that microbial growth could be causing detrimental effects at A-6A. An independent evaluation recommended that IRB were the principal cause of the decline in well specific capacity (Black and Veatch, October 1993). Based on this recommendation, a comprehensive well rehabilitation project was formulated to address the issues described above, ranging from controlling the biological activity, to addressing encrustation on the screen, to redeveloping the well to remove any chemical precipitates from the screen section and/or the gravel pack.

After completion of the well rehabilitation effort, the well specific capacity, as measured by the subsequent pump test, was virtually unchanged from the pump test conducted just prior to the well rehabilitation effort. This indicates that none of the potential factors that could cause problems with the injection process, including growth of IRB, affected the well specific capacity at A-6A

Efficiency – The efficiency of the injection process was measured in two ways: (a) the well efficiency as measured by the step-drawdown tests and (b) the ratio of the amount of water that can be injected to the amount of water that has to be pumped during the pump cycles to maintain the well at optimum efficiency.

Step-discharge tests were conducted routinely as part of each pump cycle. The well efficiencies were maintained in the 80+ percent range. This indicates that there was little detrimental effect to well A-6A as a result of the injection process, even with the ever-increasing injection rates.

The other measure of efficiency of the injection process was to evaluate the volume of water injected versus the volume of water that had to be pumped to maintain the injection process without clogging the injection well, or otherwise causing detrimental effects to either the well or to the aquifer. A total of 1283 ac-ft (5.2×10^6 m^3) was injected as part of the project, with a corresponding 22.4 ac-ft (9.1×10^4 m^3) of water being pumped during the pump cycles. This yielded an average injection efficiency of 98.3 percent.

Solids Production – Sand production was monitored in each of the pump cycles by Rossum meters. No sand production was observed during the duration of the Demonstration Project. However, small flakes of iron oxide were typically observed in trace amounts in the Rossum meters.

Water Quality

The native Arapahoe aquifer water in the vicinity of A-6A is calcium-bicarbonate-type water. With the injection of Denver water, which is a predominantly calcium-sulfate to calcium-bicarbonate water, the major ion concentrations shifted to more closely resemble that of the source water.

Even though there was a shift in water quality from the native Arapahoe aquifer water quality to a quality that was more chemically-similar to the Denver source water, no adverse chemical reactions were observed during the course of the project that would cause concerns related to the injection process.

Economic Evaluation

Even assuming the technical viability of a full-scale injection, storage and recovery project, it was necessary to also evaluate the economics of the process to determine if this conjunctive use program could be operated in a manner that would make it competitive with other water supply sources. One of the principal advantages of a conjunctive use plan, as conceptualized, is that existing facilities would be used, and that these facilities would be used during off-peak times, thereby obtaining better utilization of existing facilities that would either be idle, or operating at reduced capacity. This presumption was used in the economic analysis.

In evaluating the cost for an injection, storage and recovery project, it was assumed that existing production wells would provide potable water supply to municipalities and, therefore, the cost of the well, pump, pump house, etc. would have been incurred regardless of whether an injection, storage and recovery project was initiated. In addition, it was assumed that the existing pumping well would be capable of a retrofit to allow both pumping and injection to occur through the same structure, i.e. the casing diameter would be sufficient to allow the installation of injection piping and a pump column. As such, water produced from a conjunctive use project would be deliverable to consumers within the existing infrastructure and would be of potable quality.

The costs related to an injection, storage and recovery project were evaluated in three main categories, (a) the cost to purchase the water, (b) the initial cost to retrofit a well to allow injection, (c) the annual cost to inject water; and (d) the annual cost to recover water.

The costs related to these factors were developed from site-specific data obtained during the Demonstration Project in conjunction with a number of assumptions. The assumptions used in estimating the injection/recovery costs include (a) the injection retrofit equipment would have a minimum useful life of 20 years, (b) the annual cost of the injection water is Denver Water's wholesale rate (\$1.65 per 1,000 gallons or \$0.43 per m^3), (c) the labor costs and water quality testing costs for a full-scale project would be approximately one-half the costs of the research and development project, and (d) the pumping costs would be similar to current electrical costs for pumping an Arapahoe aquifer well.

Based on these assumptions using 1997 prices, the total cost for delivering water to the injection well, injecting it into the Arapahoe aquifer, storing it in the aquifer and then recovering it for subsequent use, is approximately \$2.45 per 1,000 gallons (\$0.65 per m^3).

Institutional Issues

At the initiation of the project, water law in the State of Colorado for the nontributary bedrock aquifers of the Denver Basin was governed by Senate Bill 5 (C.R.S.37-90-137). In that statute [(C.R.S.37-90-137) (4) (b) (ii)], it states that "the amount of such ground water available for withdrawal shall be that quantity of water, <u>exclusive of artificial recharge</u>, underlying the land owned by the applicant" [emphasis added]. Therefore, it was clear that Colorado statutes did not allow credit for use of water introduced by artificial recharge and, therefore, a new regulatory framework needed to be developed that would allow a Denver Basin aquifer storage and recovery system to operate under Colorado water law.

To develop a technical data base to assist in formulating the regulatory framework, parametric studies were conducted to evaluate injection and recovery criteria. The variables that were assessed as part of the parametric studies included the timing of injection and pumping, aquifer transmissivity, pumping rate, spacing between wells, aquifer condition (confined, semi-confined or unconfined) and whether the water was recovered from the same well that the water was injected into, or from a remote well.

The Denver Basin Artificial Recharge Extraction Rules were developed and promulgated on July 1, 1995. Generally, the Extraction Rules allow the storage of artificially-recharged water in the Denver Basin aquifers for an unlimited period of time, with 100 percent of the water injected allowed to be extracted. The artificially-recharged water can be extracted either through the point of injection or at points remote from the injection point, up to 5 mi. (8.05 km) distant from the injection point as long as there is a contiguous parcel. Remote extraction from a non-contiguous parcel is also envisioned in the Extraction Rules (Rule 10.1), however, this requires providing site-specific data to the SEO to demonstrate lack of injury to intervening water rights.

CONCLUSIONS AND RECOMMENDATIONS

Based on the data generated during the course of the project the following conclusions and recommendations have been reached relative to the implementation of a full-scale deep well injection, storage and recovery project:

(1) A full-scale injection, storage and recovery project in the Denver Basin is technically, economically and institutionally feasible.

(2) It is important to understand the difference in injectant temperature from the native aquifer water temperature so that viscosity changes can be accounted for in the operational aspects of the project.

(3) Chemical compatibility studies should be completed for the source water and aquifer water, including analyses of the formation matrix to assess potential chemical or geochemical problems prior to the initiation of a full-scale project.

(4) The efficiency of the injection process is maximized by the periodic flushing of the well through pumping. Therefore, a cyclical process of injection and pumping is recommended. However, this pumping generally only represents 1 to 2 percent of the water injected.

(5) It is extremely important to control the downhole flow through the injection pipe so that no entrained air is allowed into the formation as a result of the injection process. This is done by maintaining positive pressure on the injection pipe at the upstream end and keeping the injection pipe submerged at the downstream end.

(6) The well screen design and the aquifer hydraulic characteristics are very important to understand as they may greatly influence the ability to inject water into the formation.

(7) Well completion is an important factor in the overall success of an injection project, as the casing needs to be of sufficient diameter to accommodate injection/pumping piping, the well should be gravel-packed to stabilize the formation, and engineered screens should be installed to maximize flow efficiencies.

ACKNOWLEDGMENTS

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GROUND WATER IN THE TURKEY CREEK BASIN OF THE ROCKY MOUNTAIN FRONT RANGE IN COLORADO

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ABSTRACT

Evaluation of front-range fractured aquifers is difficult because the expense of characterization is not deemed warranted for development decisions. Data integration in Turkey Creek Basin, a well-studied area, reduces uncertainty and eventually will identify the key data required for characterization. Current analysis of the available data reveals the basin can be represented with an equivalent porous media model to facilitate management decisions at the watershed scale. However, impacts on individual wells cannot be predicted accurately. Water levels are declining and water quality is impacted by anthropogenic activity in Turkey Creek Basin, but the available data only provide an estimate of whether the basin can sustain the current population. Using one approach, annual recharge is estimated to be on the order of an inch per year (25.4mm/yr), with 75% of that volume pumped, but only 7% consumed. However, the estimates are uncertain due to the short period of record and limited spatial distribution.

INTRODUCTION

Development in the Colorado front-range is increasing, warranting assessment of ground-water conditions to evaluate the magnitude of population that these basins can sustain. The Turkey Creek Basin (TCB) in Jefferson County, Colorado is experiencing rapid growth. This semiarid watershed is about 20 mi (32 km) west of Denver (Figure 1), encompassing approximately 47 square mi (122 km^2) and ranging in elevation from 6,000 to nearly11,000 ft (1800-3400 m). More than 11,000 residents are served by the fractured-crystalline rock aquifer providing water through individual domestic wells, with the overlying regolith treating water from sewage disposal systems.

Typically, fractured aquifers are only well characterized at sites where substantial funds are available, such as potential nuclear waste repository locations. A more cost effective way of characterizing fractured aquifers is necessary for water supply problems. Surface investigation of fractures has limited applicability to aquifer characterization because the distribution and size "or aperture" of fractures in outcrops are not directly related to the distribution, or productivity, of water-bearing fractures in the subsurface. Borehole televiewer and flow meter logs clearly show that only a few of the observed fractures produce water in the subsurface (Folger, 1995).

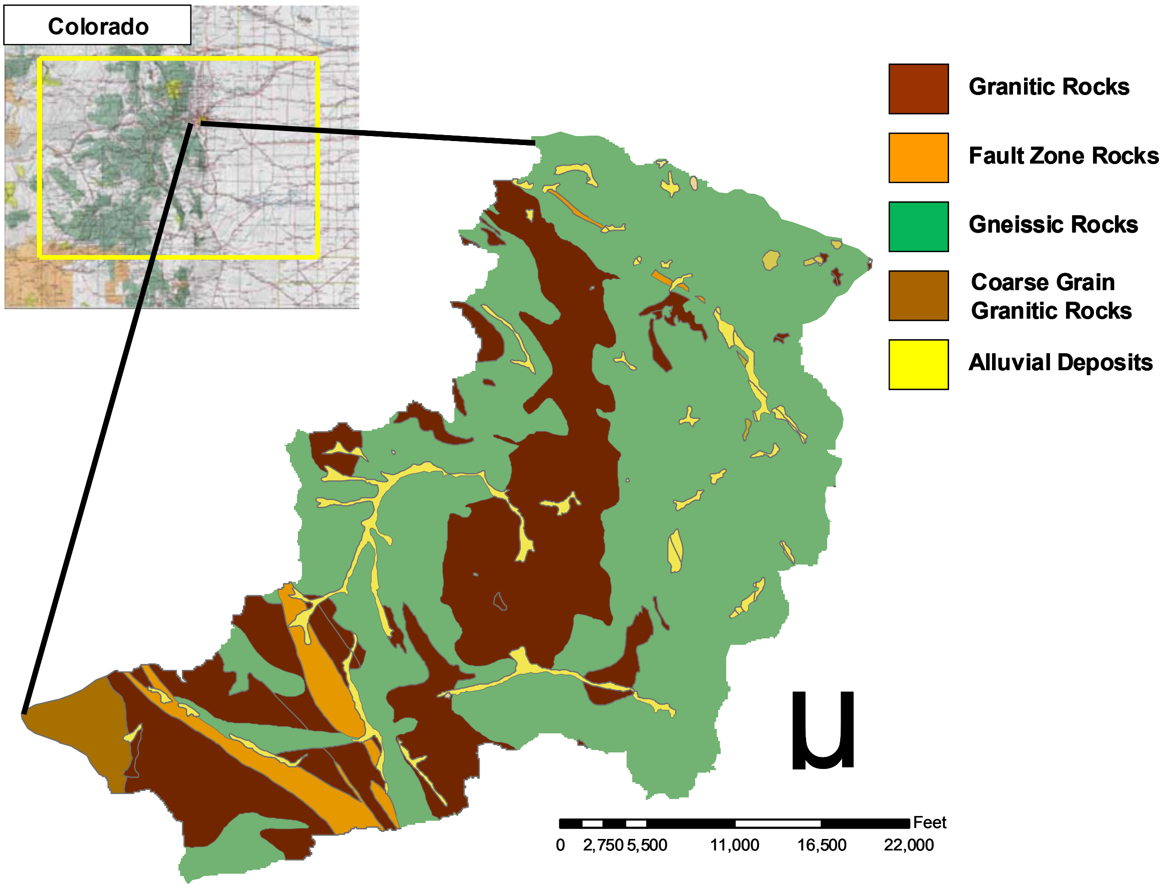

Figure 1. Location and geology of Turkey Creek Basin (after USGS 2001)

Often valuable data are collected and lost with time. Integration of data from many disciplines, sources, and time periods enhances the practice of engineering geology. This project locates, and integrates, typical watershed data to characterize the hydrology of the Turkey Creek Basin and evaluate its water resources. Historical data are available from a Colorado Geological Survey evaluation of ground water in the 1970's and from the State Engineer's Office since the 1930's. Hydrologic and fracture data are gleaned from more than 1100 driller's logs. Recent evaluations conducted by Jefferson County, in collaboration with the U.S. Geological Survey, provide additional data, and ongoing field measurements enhance the analysis. Local water districts, particularly the Indian Hills Water District accumulated data for decades and freely shared their knowledge. Consulting companies have collected data, and once identified, the companies share data if their client so requests. A database is under development with periodic updates available on the web (see Appendix). Data holders willing to share information are encouraged to contact the authors. Although many of the data are of relatively low quality, the large volume provides qualitative insight on spatial and temporal trends of system parameters.

This paper reviews the geohydrologic data for Turkey Creek Basin; presents the water budget, noting the associated uncertainties; and evaluates the water chemistry. Uncertainty in the data and evaluation are discussed. The conclusions outline current understanding of water resources in TCB. Readers who are not interested in the supporting data may want to proceed to the section titled “Water Budget”.

GEOLOGY

Geologic Units

The major rock groups were delineated by Bryant (1974) and combined into four primary groups (Caine, 2001; USGS, 2001), Figure 1. The four groups include: 1) Coarse granite (Pikes Peak granite, equigranular biotite-hornblende granite, outcropping only in the southwest portion of the basin); 2) Gneiss (gneiss and schist); 3) Granite (intrusive quartz monzonites and other granitic rocks); and 4) Fault zones (occurring in all groups). All units are composed of the same minerals, which include potassium-feldspar, plagioclase, quartz, biotite, muscovite, and hornblende. Regolith (weathered rock) forms a thin veneer over most of the basin and limited alluvial deposits occur beneath the streams.

Water-bearing Fracture Distribution

Frequency of water-bearing fractures is nearly uniform between rock groups, while local variations are substantial. Frequency was determined by counting the water-bearing fractures, noted by drillers on 1117 well logs, over the depth interval of interest. Intervals were defined from the water table to the next round-100 ft depth, then each 100 ft interval there after. The number of fractures in a given depth interval for all wells was divided by the total number of feet of borehole drilled in that depth interval. Average frequency is ~0.01 fractures per foot for the four major geologic units. The frequency was above average (~0.012) in the first 100ft (30m), then constant (~0.008) to a depth of approximately 700ft. Lower fracture frequencies occur in the fault zones and coarse granite (Figure 2). However, total well yields are higher in the fault zones and coarse granite indicating either under-reporting of fractures in those rock types, or fractures in these units have larger apertures and/or better connectivity. The more productive units have limited spatial extent in the high elevations of the southwest corner of the basin (Figure 1). These statistics may be biased by legal implications associated with reported well yield, which may lead to incorrect recording of yields. For example, there is a limit of 15 gallons per minute (GPM) (57 liters/minute, LPM) for domestic use and some general opinion that 2 GPM (8 LPM) is a minimum for obtaining a home mortgage (9% of the reported yields were 15 GPM and 5% were 2 GPM).

HYDROLOGY

The principal aquifer in Turkey Creek Basin is comprised of fractured-crystalline rocks to a depth of at least 900 ft (274 m), which is the greatest depth drilled to date. Most of the development in TCB occurred in the last few decades as documented by well completions and first beneficial uses reported in well records (Figure 3a).

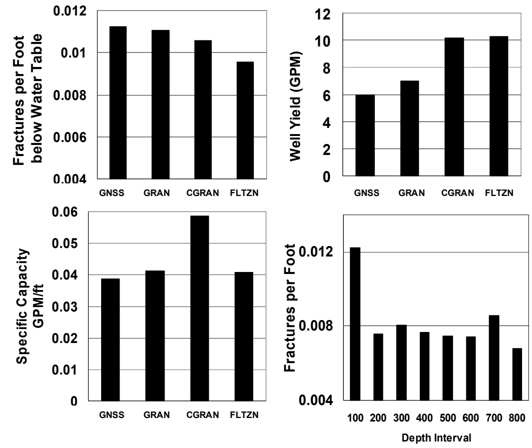

Figure 2. Hydraulic character of geologic units in Turkey Creek Basin: a) fracture frequency by rock group; b) well yield by rock group; b) specific capacity by rock group; and d) fracture frequency by depth interval (metamorphic gniess and schist, GNSS; granite, GRAN; coarse granite, CGRAN; and fault zones, FLTZN).

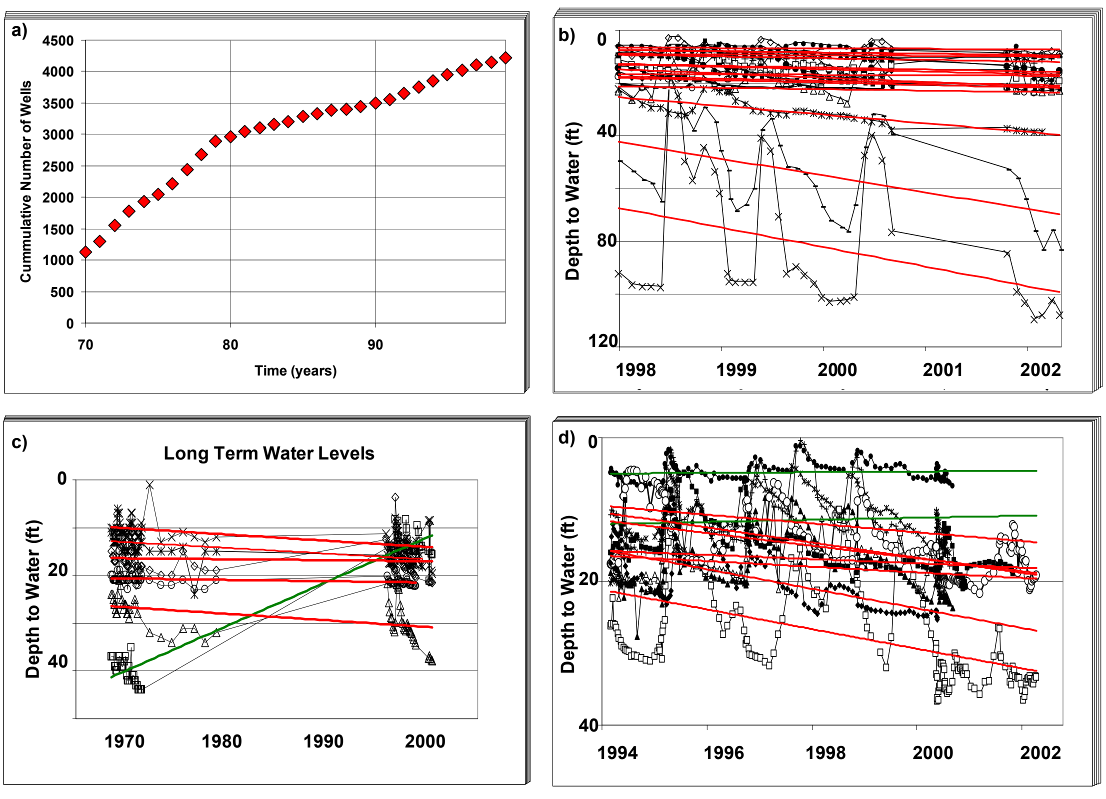

Figure 3. TCB wells and monitor-well water level trend lines (Red lines indicate decreasing water level, Green Lines indicate rising water level); a) number of wells in TCB; b) trend lines of monitor-well hydrographs in recent years (USGS, 2002; Hatch, 2002); c) trend lines of hydrographs in six wells with measurements over 30 years (USGS, 2002; Hatch, 2002); and d) trend lines of hydrographs in Parmalee Gulch from 1994 to 2002 (IHWD, 2002). Note scale changes between figures.

Water Levels

Contours of the water levels from the Colorado State Engineer's Office (SEO) well logs and USGS measurements (Bossong, 2002) mimic topography, revealing a ground-water basin that coincides with the surface water basin, and discharge of surface and ground water is focused in a narrow canyon in the northeast (Figure 4). Analysis of 6900 well records from the SEO indicates average depth to water is less than 100ft (30m).

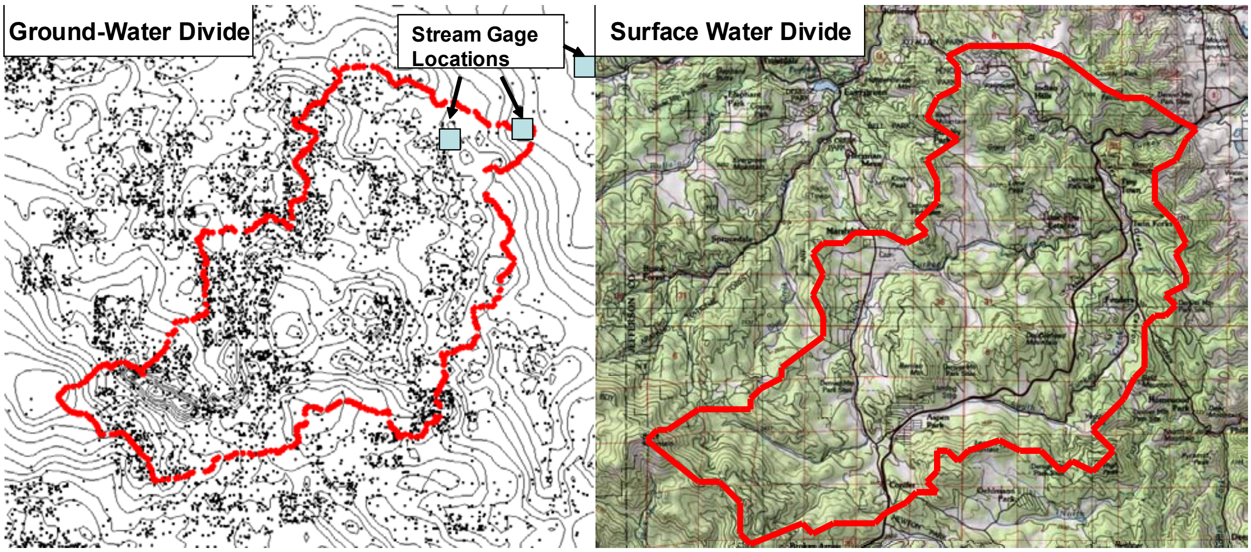

Figure 4. Head distribution in TCB showing the water table mimics topography, and the ground-water divide is coincident with the topographic divide. Heads are approximate because they are determined using the reported static level and elevation extracted from a digital elevation map at the reported x-y location (both water level and x-y location are commonly in error on well logs) at the time of drilling (thus they do not represent levels of the water table at the same time). The small plus signs indicate the locations of wells used to generate the head contours. The bold outline delineates the basin boundary.

Monthly water level measurements in 15 unpumped wells during recent years indicate: water levels are responsive to seasonal recharge to varying degrees; and overall, levels are declining (Figure 3b). Water levels in the five wells with measurements from 30 years ago that exhibit water level decline average a drop of 1 ft/yr (0.03 m/yr) from 1973 to 1998 (Figure 3c). One well exhibits a large rise in water level since the 1970s, likely due to a change in the local pumping regime. All fifteen wells measured during the past four years reveal a higher average decline of 1.3 ft/yr (0.4 m/yr) from 1998 to 2001 (Figure 3b). A water district in a sub-basin of Turkey Creek (Parmalee Gulch) where the county implemented development restrictions in the 1970s provided water levels from a number of wells from 1994 through 2002 (Figure 3c). Two of the 9 wells show a slight rise in water level while one exhibited no change and the other 6 declined at an average rate 0.5 ft/yr (0.15m/yr) less than that of the fifteen monitoring wells throughout the basin. This may reflect the effect of development restrictions. The recent increase in the rate of decline in other areas of the basin may be a response to short-term climatologic conditions or reflect water levels that have not reached equilibrium with the current usage. A longer period of record is required to distinguish the cause.

Shallow water levels that mimic topography coupled with rapid response to precipitation and snowmelt indicate the fractured bedrock is a relatively well-connected, unconfined aquifer.

Stream Discharge

Stream discharge records for Turkey Creek are limited and complicated by shifting station locations (Figure 3) and diversions. Turkey Creek was gauged for short periods in the 1940's and 1980's at USGS gauge 06711040 (Figure 5a and 5b). Later (April 1998 to April 2001), it was gauged at 06710995 (Figure 5c), approximately 2 mi (~3 km) upstream from 06711040. However, that site was unstable due to diversions (Bossong, 2002), so the current gauge was established in April 2001 as 06710992 (Figure 5d). It appears that stream flow is larger at the down gradient location, but difficult to know if this is due primarily to differences in precipitation during their years of measurement (Figure 5e). There were 6 days of coincident discharge measurements at 06710992 and 06710995 (Figure 5f) exhibiting a reasonably consistent relationship, but a longer period of overlap is required to estimate historical flow at 06710992 because the regression equation (Figure 5g) produces a minimum discharge of 0.726 ft^3/sec (cfs) (0.02 m^3/sec (cms)) at 06710992, which is frequently dry. Given these limited data, it appears that Turkey Creek gains as it exits the foothills and enters the plains, but it is difficult to know whether the gain reflects upwelling of ground water recharged in the mountain basin, or contributions from lowland areas near the mouth of the basin. Based on these limited data, baseflow is estimated on the order of 1 cfs (0.28 cms). Although, arguably, it could be less, perhaps being as low as 0.3 cfs (0.0085 cms). Effective uniform depth (EUD) is a measure of volume in terms of a depth spread over a basin area. It is a useful measure for comparison of water budget items to annual rainfall. One cfs is an EUD of 0.3 in/yr (0.76 cm/yr) in TCB, whereas a baseflow of 0.3 cfs is an EUD of 0.09 in/yr (0.23 cm/yr).

Precipitation and Evapotranspiration

Precipitation records from the USGS (USGS, 2001; Bossong, 2001, 2002; Stannard, 2002) are available for a short period (1999 through most of 2001) from 7 tipping buckets and 4 weighing buckets. Evapotranspiration measurements are available from an eddy-correlation tower over a forested area and a Bowen-ratio tripod evapotranspiration (ET) station in a meadow, (both in the central basin). Annual precipitation during the period is on the order of 20 inches (508 mm) per year (Table 1 and Figure 5e). This is high compared to long-term regional records, which indicate 17 to 18 in/yr (432 to 457 mm/yr) (Hansen, Chronic and Mattlock, 1978). Denver precipitation was higher than the average 15.8 in/yr (402 mm/yr) in 1999 and 2001 (~5 in (127 mm) and 0.75 in (19 mm) higher respectively), and slightly lower (1.25 in, 32mm) in 2000 (NOAA, 2003). Similar trends may be expected in TCB. The average precipitation at the ET measurement sites is higher than the average of all stations in the basin by 2 in (51 mm) in the two years of complete records. Clearly, a longer record is necessary to determine whether these stations are in areas of consistently higher precipitation, but the USGS discontinued monitoring at all except the ET tower location. A unique program (CoCoRaHs, see Appendix for Internet address) of precipitation recording, implemented over portions of Colorado by the state climatologist, began operation in TCB in 2002.These data will better define the precipitation distribution after several years of data have been collected.

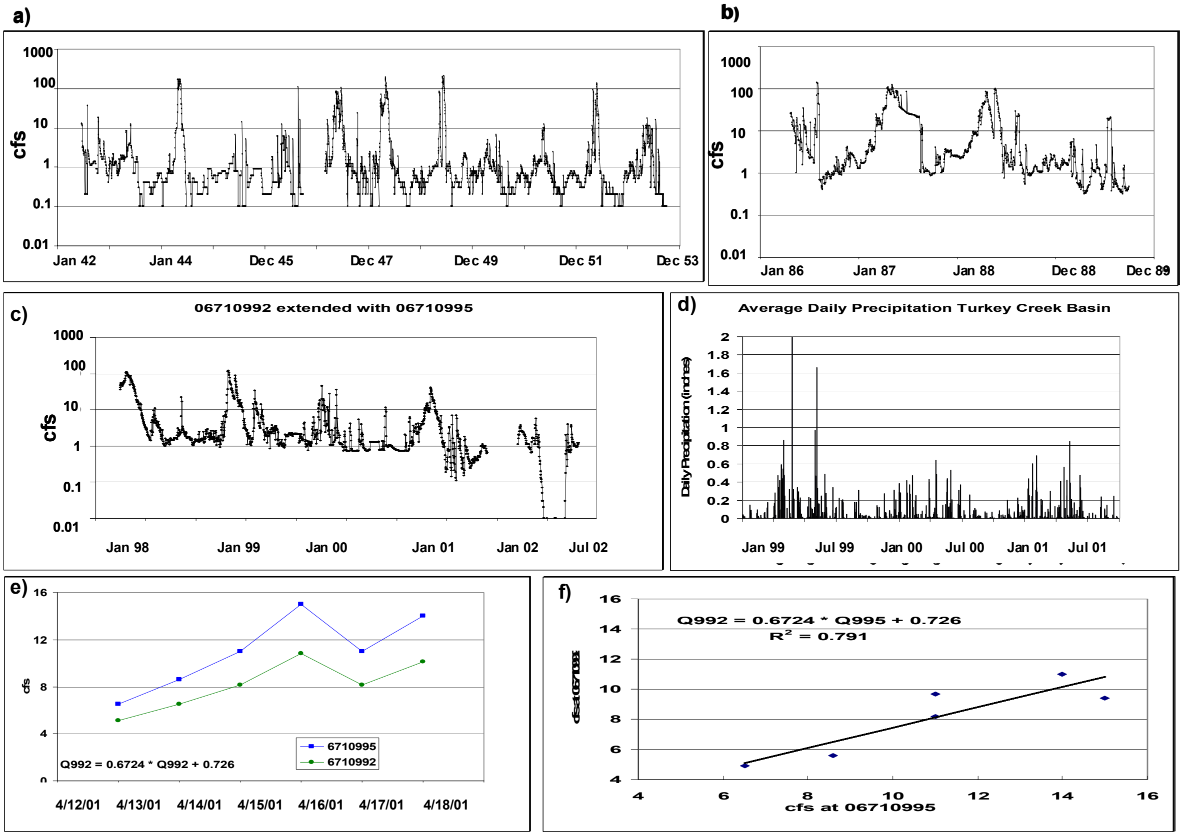

Figure 5. Stream discharge from Turkey Creek: above Bear Creek Lake in a) the 1940s, 1950s and b) 1980s (USGS gage 06711040); c) near Morrison from April 1998 until April 2001 (USGS gage 06710995) corrected for diversions (Bossong, 2002) and extended (without a correction for location) with the data form the current gage near Indian Hills, after moving the gage one mile upstream above troublesome diversions that made development of the stage discharge curve near Morrison difficult, from April 2001 to Jan 2003 (USGS gage 06710992). Missing data are caused by periods when the stream is frozen in winter, or is dry in summer. Precipitation 1999 through 2001 is presented in d). Discharge at both gauges during a 6 day period when both were operating e); and f) linear relationship for the 6 day period is likely not useful at low flows because it predicts that the flow at gage 06710992 will never be less than 0.726cfs (i.e. when flow at 06710995 is zero) while the record from clearly shows it goes to zero.

Evapotranspiration is less variable with time than precipitation, thus the percent of precipitation lost to ET can vary substantially depending on yearly precipitation. For instance, ET at the tower, which is a high precipitation location (Table 1), is as low as 71% of precipitation in a wet year (1999), and as high as 95% in a dry year (2000). Comparison of the two ET stations indicates we do not necessarily expect more ET at the location with higher precipitation. This is important because, depending on how the precipitation values are averaged over the basin, some years may have ET rates that are greater than precipitation. This may be the case in dry years. For example, the only stations with a full calendar year of precipitation record for 2001 are the ET sites. The average precipitation at the ET sites is higher than the average of all other stations by 2.05 in (52 mm) for the 2 years when complete records are available from the other sites (1999 and 2000). The partial records from 2001 confirm this difference, so an average annual precipitation for the basin that is 2.05 in (52 mm) less than that measured at the ET sites could be justified (17.15 in (436 mm) rather than 19.2 in (488 mm) for 2001). However, use of this adjusted value produces ET that is 107% of precipitation, which could not be supported on a long-term basis and is not a reasonable reflection of a wet period, thus the period of record must be too short to provide an accurate average. Issues of this nature indicate the difficulty in using short-term data of limited and variable spatial distribution. Appropriate uncertainty must be included in budget estimates.

Table 1. Local precipitation and evapotranspiration data

Precipitation (in/yr)	**Station**	**1999**	**2000**	**2001**
	rf1	22.85	18.48	
	rf3	22.11	13.93	
	rf4	23.46	16.83	
	rf5	19.86		
	rf6	21.55	10.31	
	rf7	18.75	15.26	
	rf8	25.51	15.33	
	rf9	21.94	15.67	
	rf11	26.03		
	tower	26.43	18.17	19.20
	meadow	23.46	17.35	19.20
Avg Precip All Sites		**22.90**	**15.70**	**19.20**
ET (in/yr)	**tower**	18.73	17.34	16.83
ET as % Precip at tower		71%	95%	88%
	meadow			20.56
ET as % Precip in meadow				107%
Average ET		**18.73**	**17.34**	**18.70**
Avg ET % Avg Precip		**82%**	**110%**	**97%**

Hydraulic Properties

From measurements of fractures on outcrops, Bossong and others (2003) estimate shallow porosity in granite, gneiss, and schist is on the order of 0.002%-0.003%, while shallow fault zone porosity is 0.27%. However, these porosity values are not consistent with higher storage coefficients obtained from pump tests.

Current small-scale multi-well aquifer testing of 13 pairs of wells has not produced a response in observation wells. These tests are performed in residential wells in cooperative neighborhoods, where residents agree to minimize water usage starting in the evening and lasting until late afternoon the following day. Maximum discharge for these tests is limited to production from a household hose, in our cases ~3-8 GPM (11 to 30 LPM)), duration was less than 8 hours, and well separations ranged from 75 to 600 ft (23 to 183 m).

Drawdown versus time, in the pumped wells, were used to obtain estimates of transmissivity ranging from 2 to 30 ft^2/day (0.2 to 3 m^2/day), with a geometric mean of 7ft^2/day (~0.7m^2/day), but reasonable estimates of storage coefficient cannot be obtained from drawdown in a pumping well. Assuming that the saturated thickness is the difference between the initial water level and the bottom of the well, effective hydraulic conductivities estimates range from 0.01 to 0.3 ft/day ($3x10^{-3}$ to $9x10^{-2}$ m/day), with a geometric mean of 0.03 ft/day ($9x10^{-3}$ m/day). These values fall at the low end of the range of anticipated hydraulic conductivities for fractured igneous and metamorphic rocks (Freeze and Cherry, 1979).

It was anticipated that the fractured character of the rocks would produce no response in some observation wells and rapid response in others. Interconnection was expected in at least some of the 13 pumping-well/observation-well pairs due to the gradual spatial changes of head and water chemistry (discussed later), coupled with the diverse orientation of three major fracture sets and their fracture frequency. Major fracture trends occur in the NW-SE and NE-SW with dips generally between 60 and 90^o, and in horizontal orientations. Fracture lengths range from meters (Caine, 2001) to kilometers (Hicks, 1987). Caine's (2001) data indicates frequencies on the order of 1 fracture per foot (5 per m). Given the likelihood of fracture interconnection between at least one set of wells, the complete lack of response from any observation well leads to the hypothesis that the media has a higher storage coefficient than Bossong and others estimate (0.002-0.003%). Drawdown of 65ft (20 m) would be expected in an observation well at a distance of 75 ft (23 m) in a material with a transmissivity of 7ft^2/day (~0.7m^2/day) and 0.002% (0.00002 fractional porosity) porosity after 8 hours of pumping at 8GPM (30 LPM). A 2% porosity (0.02 fractional porosity) would yield an immeasurable drawdown of $1x10^{-5}$ ft ($3x10^{-6}$ m) for the same conditions. Consequently, the conclusion is that storage, at least in shallow zones, is more likely on the order of 2%. It may decrease substantially with depth.

Data from a 30-day pump test conducted in the 1980s in Indian Hills was located and analyzed. Water district employees indicate well yields are higher in this area, which is why the district well field is located there. The primary test well (#10) pumped an average of 21.8 GPM (82.5 LPM), but the test was complicated by an additional discharge of 5.1 GPM (19.3 LPM), from a nearby district well (#5), during the test. The test data included water levels from 15 wells ranging in distance 220 to 750ft (67-230m) from the primary test well and 150 to 1000ft (46 to 305m) from the district well. Drawdown data are corrected for a declining water level trend observed over a period of 8 days before the test began. Evaluation of drawdown in the pumping well versus square-root of time, time to the ¼ power, and inverse square-root of time, produce the same curve shapes as predicted Theis drawdown versus the same transform of time (perhaps due to the lack of early time data), so the Theis model was used to interpret the data. The Theis solution for two superposed pumping wells (#10 and #5) represents response in the observation wells for the selected transmissivity and storage coefficients. Omitting the pumping wells, four of the fifteen wells exhibit responses that can be correlated with the stress of the test (Figures 6 and 7). These four wells range in distance 220 to 400 feet from the pumping well and are among the closest seven wells. Two of the other three close wells appear to respond for the first third of the test and then recover, so their response was assumed to reflect a different stress. The other close well showed no response. For one well (BO), early time data were available from a chart recorder. The response suggests an unconfined aquifer, although a number of possible causes may be hypothesized (Ehlig and Halapaska, 1978). Fitting a Theis curve to early and late time data produces a high transmissivity (3500 ft^2/day (325 m^2/day)), an elastic storage coefficient of 0.0065, and a specific yield of 0.01 (Figure 6). The high transmissivity is short lived as indicated by the positive deviation of the observed drawdown from the Theis curve. This test may be investigating a highly fractured zone of limited spatial extent. The remaining three wells are more consistent with one another, yielding a geometric mean transmissivity of 260 ft^2/day, (24m^2/day), and an average specific yield of 0.024. The aquifer thickness was defined as the difference between the water level and the bottom of the well, resulting in an effective hydraulic conductivity of 0.4 to 0.9 ft/day (0.1 to 0.3 m/day).

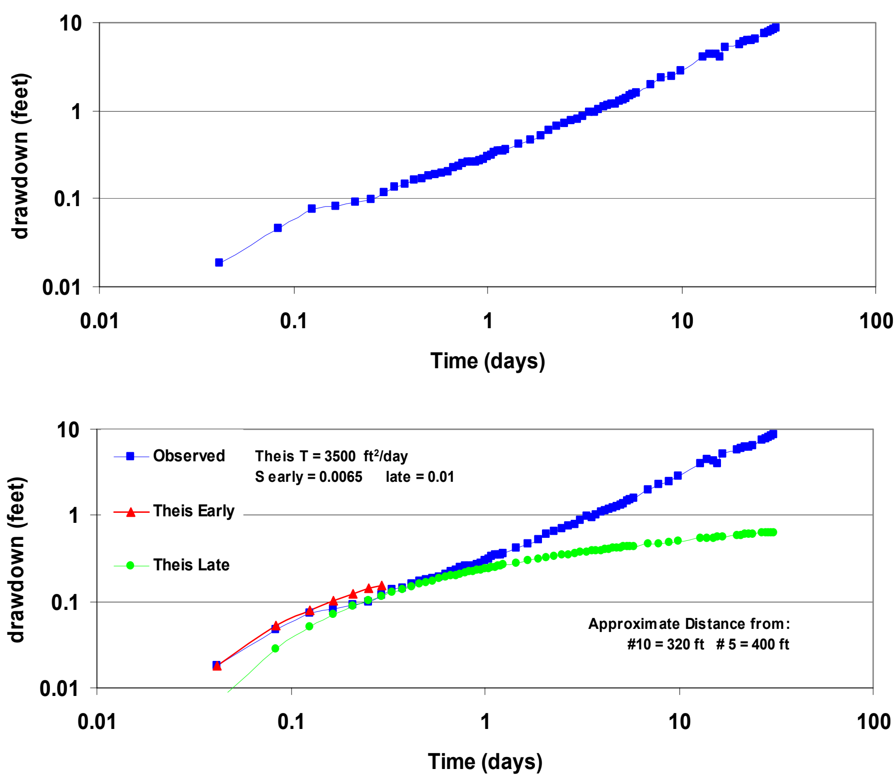

Figure 6. Theis curve fits for observation well that included early time data.

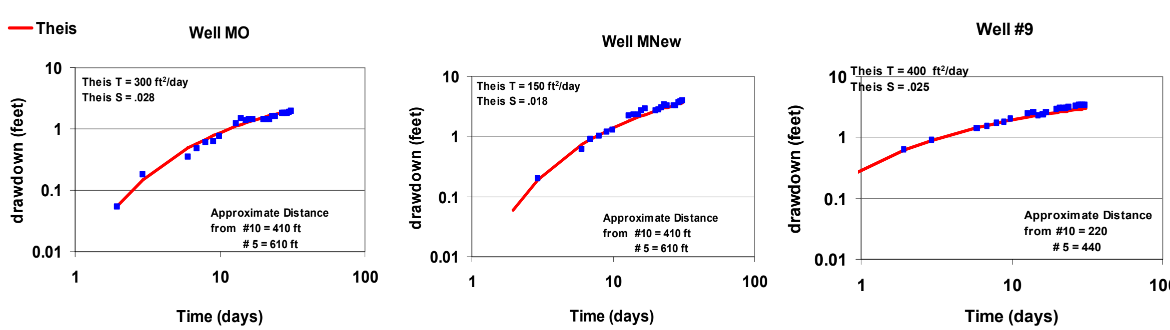

Figure 7. Theis curve fits for observation wells that exhibited a clearly correlated response to multi-well pump test, but for which only late time data were available.

WATER BUDGET

Generic Water Budgeting Considerations

Generally, water budgets are presented in a static mode representing average conditions. At times, this leads to misconceptions about response of the system to future stresses. Consideration of the dynamics of the system avoids pitfalls such as dividing the volume of water filling pore spaces by the usage rate to estimate the time that current usage could be sustained. The challenge is to determine 1) average recharge, to gage the level of sustainable consumption, and 2) estimate drought probabilities to assess the minimum recharge that is sustainable through periods of drought for communities (e.g. well density, well depth, withdrawal rates). However, complications abound, for instance, the largest water level declines will occur at wells because they are discharge points. Consequently, average water level declines are not sufficient to determine whether wells will go dry, instead local-scale analysis is necessary for that purpose.

The following dynamics are expected: given constant recharge, when pumping increases, water levels decline, the water table gradient decreases, baseflow decreases, thus stream flow and ground-water outflow decrease (Figure 8). Eventually, a new equilibrium condition is established with lower water levels and a lower ground-water gradient, such that the average ET, discharge to stream baseflow, and ground-water outflow are reduced. This is acceptable if the total stream flow is sufficient to serve downstream users and maintain an acceptable environment for aquatic species in the stream, and ground-water outflow is sufficient to maintain acceptable water levels outside of the basin in areas that receive ground-water inflow from the basin.

If a drought occurs, both precipitation (PPT) and ET decrease, but not proportionately, as PPT decreases more than ET. Drought conditions increase evaporation as well as use, so the percent of PPT lost to ET increases. Consequently, water levels, stream flow, and ground-water outflow will decrease, reducing their percent of PPT. In the extreme, if PPT ceases for a long period, all the water in the fractures and pores is not available for use, hence the problem with the simplistic volume of pores/usage rate calculation. Rather, ground water continues to flow from the basin whether wells are pumped, or not, and ground-water levels continue to decline. While the rate of decline depends on the transmissivity and storativity of the rocks and soils, water levels do not decline uniformly. Rather, they drop more rapidly at higher elevations. Consequently, gradients decrease in a non-linear fashion and the rate of baseflow to streams and ground-water flow out of the basin decrease exponentially (as observed in typical stream flow recessions in arid basins).

Development of a sustainable system involves consideration of the dynamic balance. At the extreme, an unsustainable system is one in which average consumption exceeds average recharge. Water levels will decline with occasional short-term temporary increases following recharge-events and longer-term temporary increases if average consumption does not exceed recharge in wet years. For example, if average basin-wide recharge is equivalent to 1 in/yr (25 mm/yr), it may range from 0.25 to 1.75 in/yr (6.4 to 44 mm/yr) in dry and wet years. If average basin-wide consumption is equivalent to 1.1in/yr (28 mm) spread over the basin, the basin will eventually go dry, although water levels will rise in years when recharge is 1.75in/yr (44 mm/yr). A sustainable condition requires that average consumption is less than the average recharge (Figure 9).

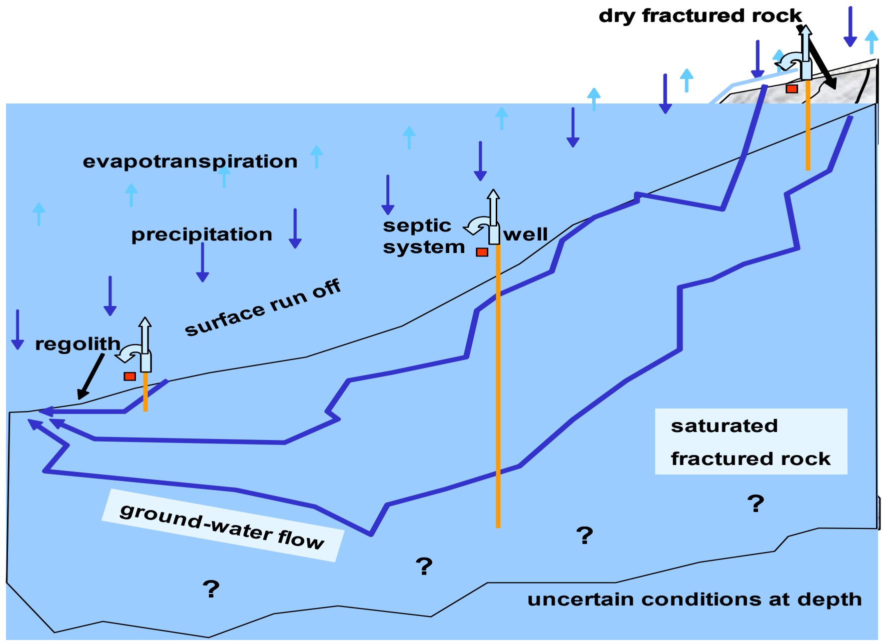

Figure 8. Envisioning the hydrologic cycle for a contained, fractured-rock basin facilitates understanding of the dynamics that need to be considered in water budgets.

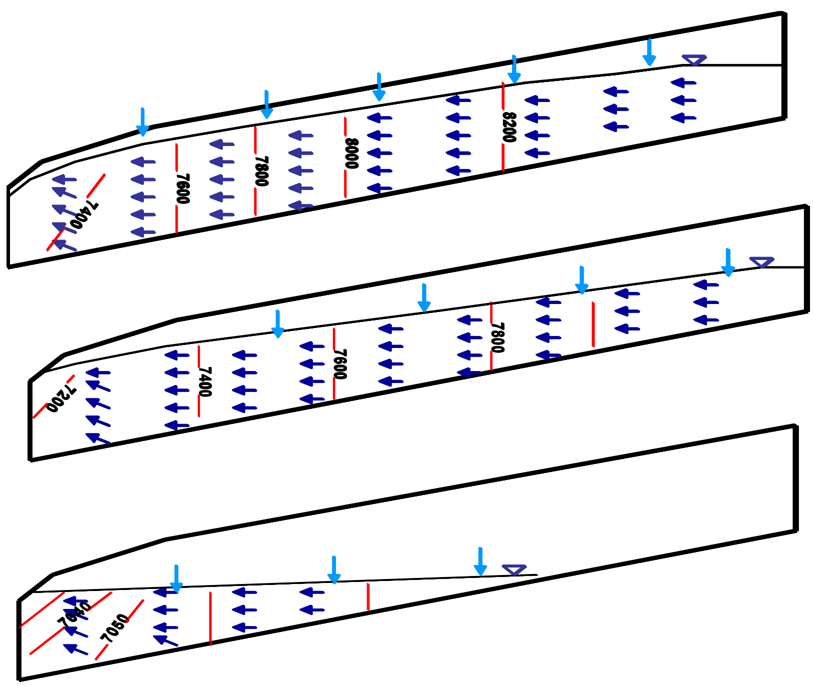

Figure 9. A cross section of a generic basin showing equilibrium hydraulic heads for cases with K = 3 ft/yr and: a) average recharge of 0.1 ft/yr; b) 50% of average recharge (i.e. doubling the population), causing head declines on the order of 250ft (76m) as controlled by the geometry, transmissivity and recharge rate (assuming instantaneous development the system reaches the new equilibrium in 1.6 years, as controlled by the storage coefficient of 0.02); and c) 10% of current recharge (i.e. increasing population by a factor of 9) nearly dries out the basin.

Turkey Creek Basin Water Budget

Geohydrologic data discussed above are combined with estimates of water consumption to develop a water budget for the basin (Table 2). The remainder, after subtracting evapotranspiration, runoff, and consumptive use from precipitation, is the combination of ground-water outflow or increase in storage. This value ranges from –1.4 in EUD/year (Table 2b using the adjusted precipitation at the ET stations for the three-year period) to 4.2 in EUD/year (in the wet year of 1999) (36 to 107 mm EUD/yr) with a "best estimate" of 0.6 in EUD/year (15 mm EUD/yr). Ground water outflow must exceed this net value because water levels are declining, thus water is leaving storage, not going into storage.

The extreme low value indicates a net loss of 1.4 in/yr (36 mm/yr), which is not reasonable for a 3-year average in a period that appears to be wetter than usual locally in spite of the regional drought. Clearly such a situation could not be sustained in the long term, so the lower bound for recharge is above this level, however, this exercise illustrates the limitations of averaging and distributing short-term data. The question of whether a net loss of 1.4 in/yr (36 mm/yr) would be consistent with the recent water level decline of 1.3 ft/year arises. However, a specific yield on the order of 0.09 would be required to yield that decline for such a net loss, and this is higher than any estimate.

An estimate of ground water outflow using maximum reasonable parameter values and Darcy's Law can be compared to net value in the water budget to evaluate whether the budget is reasonable. Given the strong correspondence of the ground and surface water divides, all ground-water outflow is assumed to occur at the northeast corner of the basin. Assuming a width of approximately 2 mi (3.2 km) for the northeast corner of the basin; a gradient of 600 ft/mile (110 m/km) or 0.11 (Figure 3); and transmissivity of 260 ft^2/day (24 m^2/day) (from the long-term multi-well aquifer test); ground-water discharge from the basin is 3.6 cfs (0.1 cms) or ~1 in (25 mm) EUD. This is a conservative maximum ground-water outflow and is more than the "best estimate", yet it is not inconsistent with the water budget (Table 2). It is reemphasized that the transmissivity value is biased because it was measured in a high well yield area and the test data indicated lower bulk transmissivity near the end of the test, likely indicating that tapped fracture system has limited extent. In addition, the width and gradient for the northeast corner are crudely estimated from a map of contoured water levels made from data taken at different times with highly uncertain, locations, elevations, and depth to water. The uncertainty in the estimated ground-water outflow extends downward a few orders of magnitude. Use of the geometric mean transmissivity from the single well pump tests, coupled with the poorly constrained gradient and flow area indicates an average ground-water discharge of only 0.1cfs (0.003 cms) or 0.03 in EUD/yr (0.8 mm EUD/yr), which would suggest either recharge is over estimated or the some intermediate parameters values for estimating ground-water outflow are nearly representative of the system. For example, a reasonable transmissivity of 155 ft^2/day (14 m^2/day) would exactly balance the budget. An extensive drilling and testing program in the area where the basin meets the plains would reduce uncertainty associated with this estimate, but the cost exceeds that which society chooses to spend for evaluating water resources.

Table 2. a) Water budget with volumes presented as effective uniform depth in inches over the basin. These are estimates associated with substantial, but unquantified uncertainty. The white rows are inflows and yellow rows are out flows. The blue row is recharge to the water table before pumping, and the gray row is the net imbalance. b) Water budget using adjusted precipitation at ET stations.

a) Calendar Years	FOREST 1999 (in)	2000 (in)	2001 (in)	Average (in)	% of Precip	MEADOW 2001 (in)	% of Precip	Distributed 75% Forest 25% Meadow (in)	% of Precip
Precipitation[1]	26.43	18.17	19.2	21.3	100.0	19.2	100.0	20.8	100.0
Evapotranspiration[1]	18.73	17.34	16.83	17.6	82.9	20.56	107.1	18.4	88.5
Pumped[2]	0.75	0.75	0.75	0.8	3.5	0.75	3.9	0.8	3.6
Wastewater Disposal (90% of pumped)	0.675	0.675	0.675	0.7	3.2	0.675	3.5	0.7	3.3
Overland[3] Runoff (80% of Streamflow)	2.74	0.72	0.66	1.4	6.5	1.37	7.2	1.4	6.6
Base Flow[3] (20% of Streamflow)	0.69	0.18	0.17	0.3	1.6	0.34	1.8	0.3	1.7
RECHARGE: Precipitation-ET-Overland	4.96	0.11	1.71	2.3	10.6	-2.73	-14.2	1.0	4.9
NET: Ground Water Out & Storage Change Precip-ET-Pumped+Returned-Ovrld-Base	4.20	-0.14	1.47	1.8	8.7	-3.15	-16.4	0.6	2.9

b)	Average (in)	% of Precip
Precipitation[4]	18.6	100.0
Evapotranspiration[5]	18.3	98.2
Pumped[2]	0.75	4.0
Wastewater Disposal (90% of pumped)	0.675	3.6
Stream[3] Spring Runoff (80% of total)	1.4	7.4
Stream[3] Base Flow (20% of total)	0.3	1.8
RECHARGE: Precipitation-ET-Overland	-1.05	-5.6
NET: Ground Water Out & Storage Change Precip-ET-Pumped+Returned-Ovrld-Base	-1.5	-7.9

[1] USGS (2002)
[2] USGS (2001)
[3] USGS stream gages 06710995(4/99-4/01) and 06710992(4/01-present)
[4] Average of all stations 1999 and 2000, average of ET stations – 2.05in
[5] 75% average forest ET station ET (1999-2001) + 25% meadow ET 2001

Recharge in Turkey Creek Basin

The TCB water budget (Table 2) illustrates that most of the precipitation is lost to ET and a substantial portion of the remainder rapidly discharges as stream flow during the spring runoff (also shown in Figure 5). Using the average rates from the short-term record for the ET stations, and considering the proportion of forest and meadow over the basin, the remaining water constitutes recharge of approximately 1 in (25 mm) EUD. The combined estimates of baseflow 0.3 in (8 mm) EUD and groundwater outflow 0.6 in (15 mm) EUD approximately equal this recharge, lending confidence to the estimate.

Of the approximate 1 in (25 mm) of recharge, 75% is pumped (1900 acre-ft per year, 2.3 million m^3/yr, (USGS, 2001), an EUD of 0.75 in, 25 mm), but much of it is returned to the subsurface via individual sewage disposal systems (ISDSs). The amount returned is uncertain but the SEO (Graham, 2003) assumes 90%, leaving approximately 0.93 in (24 mm) of EUD per year to supply stream baseflow, ground-water outflow, flow to storage, or future development. Ongoing studies are evaluating this return rate. If none of the pumped water returns to the deep aquifer, then the 1900 acre-ft/yr (2.3 million m^3/yr) would yield an average stream discharge of 2.6 cfs (0.07 cms). This is not consistent with discharge at the Turkey Creek gauge near Indian Hills, where discharge during the only complete water year of record, 2002, yielded 400 acre-ft (1.7 million m^3), which is on the order of 0.5 cfs (0.02 cms) (Figure 5). Consequently, a considerable portion of the water must be returned to the aquifer. However, when the gauge was located near Morrison, discharge was 8200 and 2200 acre-ft (3.6 and 9.6 million m^3), or 11 and 3 cfs (0.3 and 0.08 cms) respectively for calendar years 1999 and 2000. Precipitation was higher in 1999 than 2000, so the higher downstream flow indicates either a substantial contribution from runoff and tributaries between the gage stations, or a substantial gain from the alluvium as the stream gradient decreases.

The "best estimate" of average annual consumption is on the order of 4-10% of the "best estimate" of annual recharge. If less than 10% of recharge is being consumed, then the observed water level declines in the basin must indicate that 1) the system has not yet reached equilibrium with the development; and/or 2) average recharge is higher than was measured during this period and the observed water level declines reflect a dry period response.

The limited period of record and spatial coverage introduce large, but unquantified uncertainty. Recent preliminary modeling efforts suggest the forest ET tower may be in an area that receives the highest recharge in the basin. If this is eventually determined to be so, then this budget overestimates the basin wide recharge.

WATER QUALITY

In addition to assessing the volume of water available, its quality is important in evaluating the sustainable population. Excluding solutes from anthropogenic sources, solutes in ground water are derived from rock-water reactions that occur along the flow path. These reactions produce gradual, systematic increase in total dissolved solids (TDS) along flow paths in porous media. In contrast, spatially abrupt changes in water chemistry can indicate fracture flow, where the flow behavior cannot be represented as an equivalent porous medium (EPM).

Water chemistry data (342 samples) for TCB, collected by the U.S. Geological Survey (USGS, 2001), in cooperation with Jefferson County in 1998 and 1999 and by the Colorado Geological Survey (Hofstra and Hall, 1975) were evaluated. The CGS data include 20 surface and 15 ground-water samples collected between 1972 and 1975 and the USGS data include 51 surface and 256 ground-water samples collected in 1998 and 1999. Samples with charge imbalances greater than ±10% were considered in error and not utilized in this analysis. An analysis of the average total dissolved solids (TDS) from both periods suggests TDS of surface water increased by 40% over twenty-five years, while the ground-water TDS decreased by about 25% (Table 3). However, histograms of TDS values for the two ground-water datasets show that the 1975 data are not normally distributed, making use of average values inappropriate. The non-normal distribution, small sample sizes and lack of common sample sites limits the value of statistical comparisons between the 1975 and 1998-99 datasets. In spite of these limitations, the consistent trend of higher average Cl values with time for surface and ground water, and higher nitrate values in the later ground-water samples is of concern because chloride and nitrate are parameters that indicate anthropogenic influence.

The larger, recent dataset includes two sampling campaigns of 74 individual domestic water-supply wells and 16 surface water sites along Turkey Creek for a total of 180 samples (locations are shown on Figure 10), representing spring runoff (06/14/1999-06/29/1999) and fall baseflow (10/01/1999-11/04/1999). Multivariate analysis of 12 of the hydrochemical variables (specific conductance (SC), pH, dissolved oxygen (DO), Ca, Mg, Na, K, Cl, SO_4, HCO_3, F, and NO_3+NO_2) using hierarchical cluster analysis (HCA) via Statistica® (StatSoft, Inc., 2000) better describes the hydrochemistry of TCB. HCA groups samples into distinct populations (a.k.a. clusters) based on selected characteristics (in this case chemical constituents, specific conductance and pH) that may be significant in the geologic/hydrologic context, as well as from a statistical point of view (Güler et al., 2002). Prior to cluster analysis, some variables were log-transformed to more closely correspond to normally distributed data (pH, DO, and HCO_3 were not transformed), then all 12 variables are standardized as described by Güler et al. (2002).

Spring-runoff and baseflow data are classified in 12-dimensional space by HCA and presented as dendograms (Figure 11a-b). Both datasets are divided into four groups based on visual examination of the dendograms, with each group representing a hydrochemical facies. The clusters are ordered from left to right in a near monotonic increase of TDS. All the groups in the spring samples had relatively low mean TDS values, ranging from 77 to 257 mg/l. Examination of Figure 11a (spring runoff) reveals that group 1, 2, and 3 samples are exclusively composed of ground water (except one surface water sample in group 3; A04). Group 4 samples consist of only surface water except one ground-water sample, N99. Group 1 and 2 samples are distinguished from group 3 not only by lower TDS, but also by lower K, Cl, and SO_4, higher F and significantly lower Cl and NO_3+NO_2. In fact, NO_3+NO_2 (as total N) averages 4.07 mg/l for group 3 spring-runoff samples, between 7 and 19 times higher than the other groups.

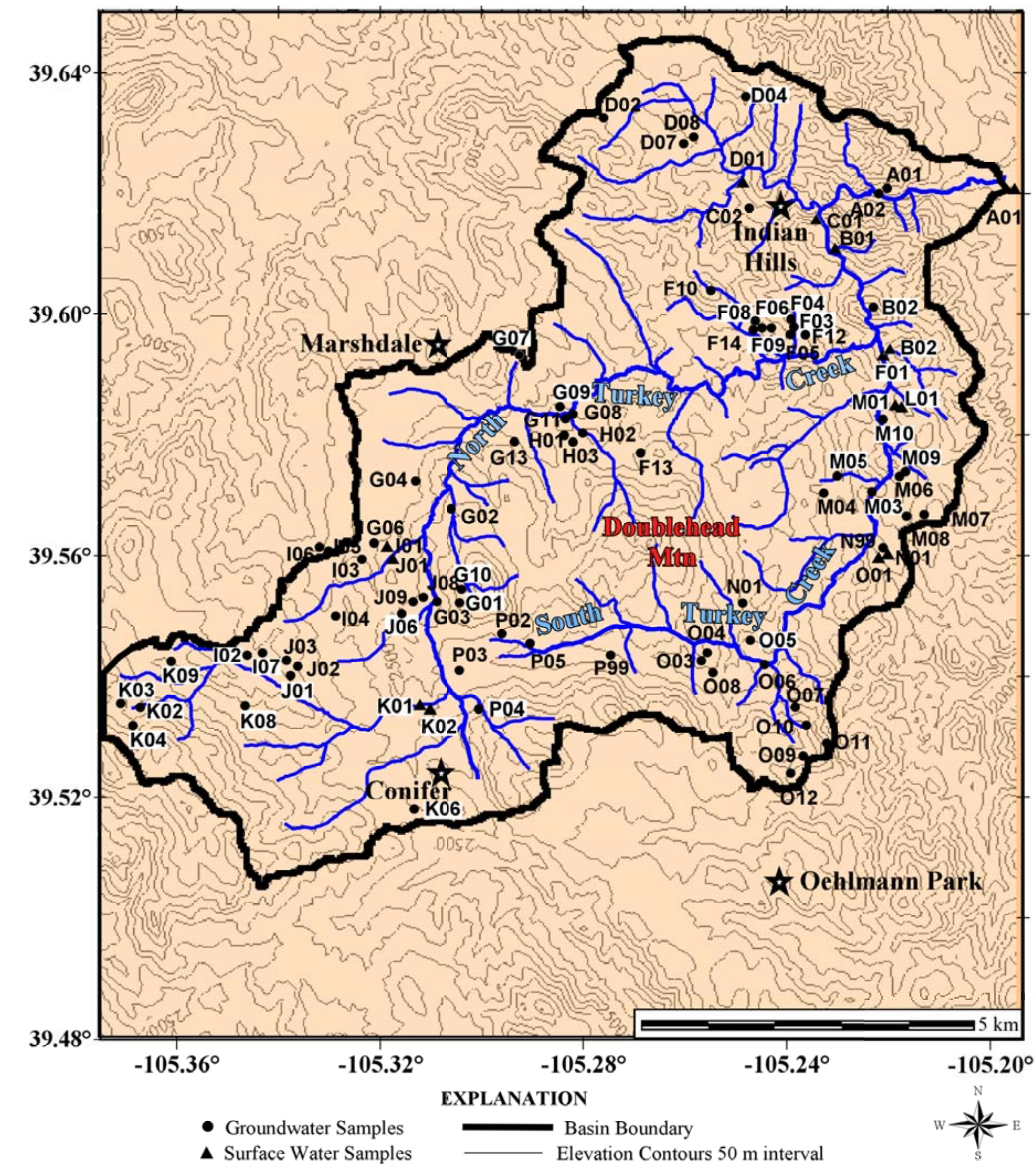

Figure 10. Locations of surface and ground-water samples.

In the baseflow samples, Group 1, 2, and 3 are composed of ground water (with the exception of surface water sample I01 in group 1; and two surface water samples in group 3; A04 and A05), while group 4 is composed of surface water samples except for one ground-water sample, N99. The means for each parameter are shown in Table 4. The grouping of baseflow samples exhibits similar trends to the runoff samples with a few differences. Mean TDS of the baseflow samples range from 95 to 286 mg/l, slightly higher than the spring samples. The TDS of group 3 is higher than group 4 due to dilution of stream chemistry during spring runoff, consistent with the

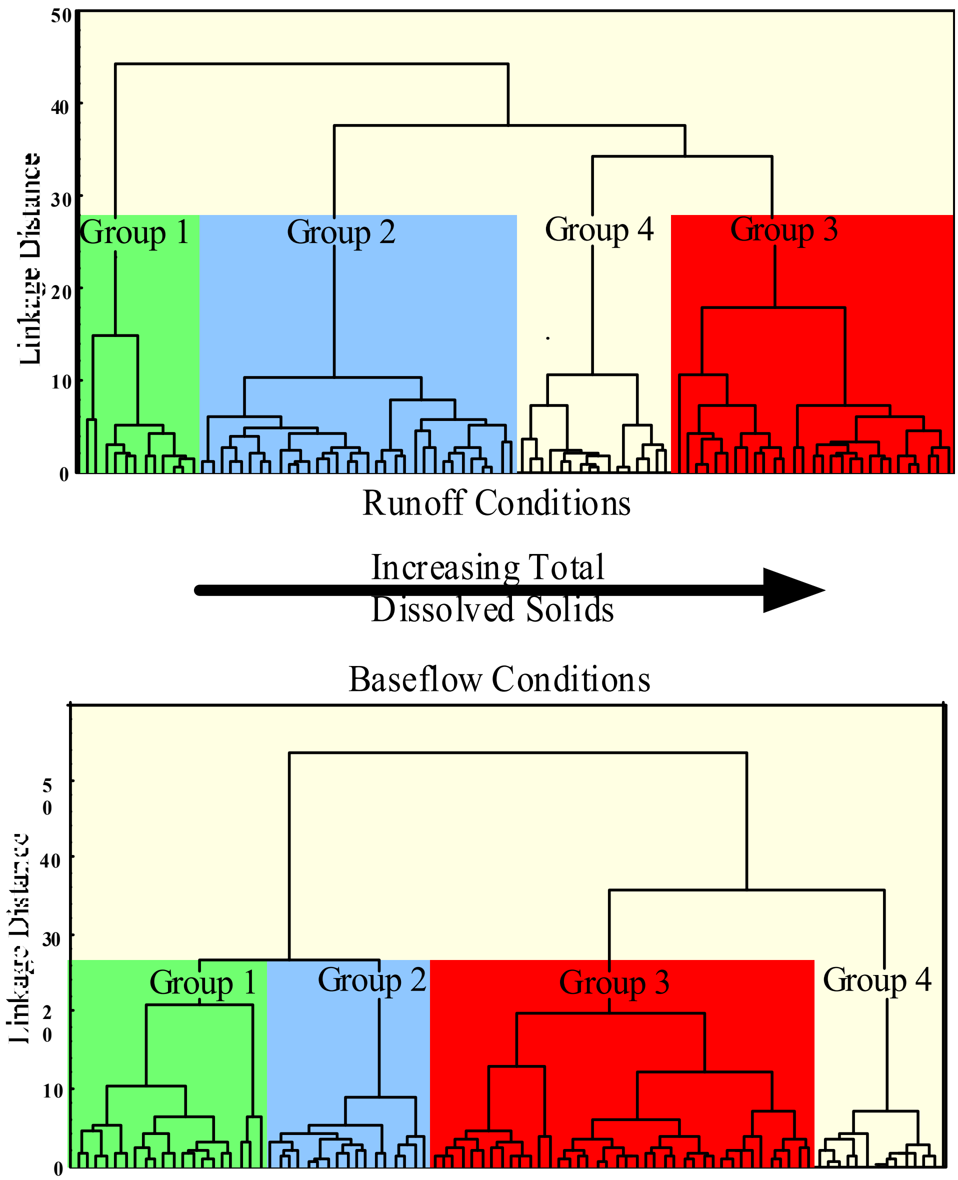

Figure 11a and b. Dendrograms of samples clustered using the HCA technique.

stream discharge data that shows most spring recharge is rapidly discharged as stream flow. In addition, the number of samples that are classified as group 3 is much larger during baseflow, indicating that during the spring-runoff period the anthropogenic influence is reduced, presumably by dilution with fresher runoff. During both spring runoff and baseflow periods, group 3 has distinctly higher Cl concentrations than group 1 and 2 and the highest NO_3+NO_2 content. During baseflow, stream water is assumed to be a result of ground-water discharge. However, the lack of chemical similarity between the ground water and surface water suggest that the contribution of baseflow to stream discharge is limited.

Table 3. Mean water chemistry of the water samples from the CGS and USGS data.

	Na	**Ca**	**Mg**	**HCO3**	**SO4**	**Cl**	**NO_3+NO_2**	**TDS**
1975 surface	12.8	30.1	6.2	89.5	36.9	12.6	0.2	237. 1
1998-99 surface	36.3	43.7	11.0	70.5	23.4	65.1	0.3	333. 8
1975 ground water	10.1	34.2	8.9	149.2	10.6	7.3	1.4	243. 8
1998-99 ground water	16.3	40.0	9.6	73.7	12.9	24.6	2.1	184. 1

Total dissolved solids (mg L^{-1}). pH (standard units), Specific Conductance (μSiemens cm^{-1}), mean concentrations (mg L^{-1}), NO_3+NO_2 as N.

Table 4. Mean water chemistry of the SPRING RUNOFF (06-1999) water groups determined from HCA

GROUP	[1]N	PH	S. C.	O_2	Ca	Mg	Na	K	Cl	SO_4	HCO_3	F	NO_3+ NO_2	[2]TDS
Group-1	12	6. 51	108. 4	5. 6	6. 4	1.8	16. 0	1.03	6. 3	7. 3	52.2	0.73	0.74	76. 9
Group-2	33	7. 47	299. 3	2.9	35. 6	9. 4	13. 6	1.17	8. 2	14.	166. 6	1.26	0.55	176. 5
Group-3	29	6. 68	478. 4	4. 0	52.6	12.0	24. 8	1.88	53. 0	30.4	142.8	0.50	4. 07	256. 6
Group-4	16	8. 17	338. 1	8. 2	30.9	7. 2	24. 2	2.52	48. 3	17	95. 8	0.65	0.21	186. 2

Mean water chemistry of the BASEFLOW (11-1999) water groups determined from HCA

GROUP	[1]N	PH	S. C.	O_2	Ca	Mg	Na	K	Cl	SO_4	HCO_3	F	NO_3+ NO_2	[2]TDS
Group-1	20	7. 03	169	6.1	13. 8	3.8	17. 8	0.69	5.9	5.9	92.4	0.55	0.47	94. 6
Group-2	17	7. 75	286.9	2.4	31.3	10.4	15. 7	1.26	4.4	9.8	175.4	1.50	0.26	162.6
Group-3	40	6. 89	503. 2	3.0	59.7	13. 4	21.9	2.25	50.9	34.2	177.5	0.48	3.13	272.1
Group-4	13	8. 00	555. 8	9.3	49.6	13. 7	36. 4	2.95	87. 8	14.1	152.9	0.61	0.28	286. 3

[1]Number of samples within groups.

[2]Total dissolved solids (mg L^{-1}). pH (standard units), Specific Conductance (μSiemens cm^{-1}), mean concentrations (mg L^{-1}), NO_3+NO_2 as N

Changes in TDS between runoff and baseflow conditions are illustrated in Figure 12. As noted above, spring runoff dramatically increases stream flow and generally lowers TDS. Locations where spring recharge lowered the TDS in wells are interpreted as areas with good hydraulic connection between the surface and aquifer. Conversely, locations with little change in TDS may indicate that vertical hydraulic conductivity is not sufficient to create the dilution effect during spring recharge. However, samples were collected over a period of a month, thus the response may have been captured by some wells and not others, adding uncertainty to the interpretation. Hydrographs from wells in TCB show that while most wells respond to spring recharge in days to weeks, some wells do not show water level increases for several months. There are a few locations where the TDS increases slightly during spring runoff. This pattern was reported in prior studies (Hofstra and Hall, 1975) and may be due to the timing of sample collection, or to spring recharge pushing older more saline ground water toward the well, although the increase in TDS is very small and may simply reflect natural variation.

Mapping sample locations according to groups illustrates the spatial distribution of water chemistry (Figure 13). Samples belonging to the same group are generally located in close proximity suggesting similar processes and/or flow-paths. Ground-water samples from group 1 are located in the higher elevations of TCB in the southwest and have the lowest TDS concentrations. Group 2 samples are located at lower elevations than group 1.The overall pattern in the basin is low TDS at higher elevations in the southwest and higher TDS at lower elevations to the northeast. The chemistry of these samples can be attributed to natural rock-water reactions and represent the hydrochemical evolution of water in the basin (Guler and Thyne, 2002). The changes in nitrate, Cl and pH between groups 1-2 and group 3 are not derived from normal water-rock reactions. Thus, Group 3 chemistry requires the input of an anthropogenic component characterized by elevated chloride and nitrate and an acidic pH. Group 3 samples correlate with higher population density, also indicating an anthropogenic component, although this correlation may be biased by the sampling pattern. The chemistry of Group 4 samples appears to be a mixture ground water discharging into the stream, itself a mixture of natural background (groups 1 and 2) and group 3 water with an additional anthropogenic component containing elevated Na and Cl.

Since the normal weathering reactions produce systematic changes in water chemistry along flow paths in porous media, we can use this feature to evaluate the flow behavior in TCB. Discounting Group 3 wells, the remaining samples show a systematic spatial distribution of water chemistry correlated with topography. This pattern, together with the correspondence of the ground-water table to topography, suggests that TCB is an equivalent porous media at the watershed scale. The influence of an anthropogenic component in the Group 3 and 4 samples is clearly distinguished, (i.e. the high nitrate and chloride). The chloride concentrations do not appear to fluctuate significantly throughout the year. This trend suggests that the anthropogenic component is primarily derived from a source such as ISDSs that has a fairly constant rate of discharge throughout the year rather than the application of road salt, which is seasonal. Elevated nitrate concentrations in group 3 and 4 samples also support an ISDS source as alternative sources such as large-scale fertilizer applications and animal operations are not present.

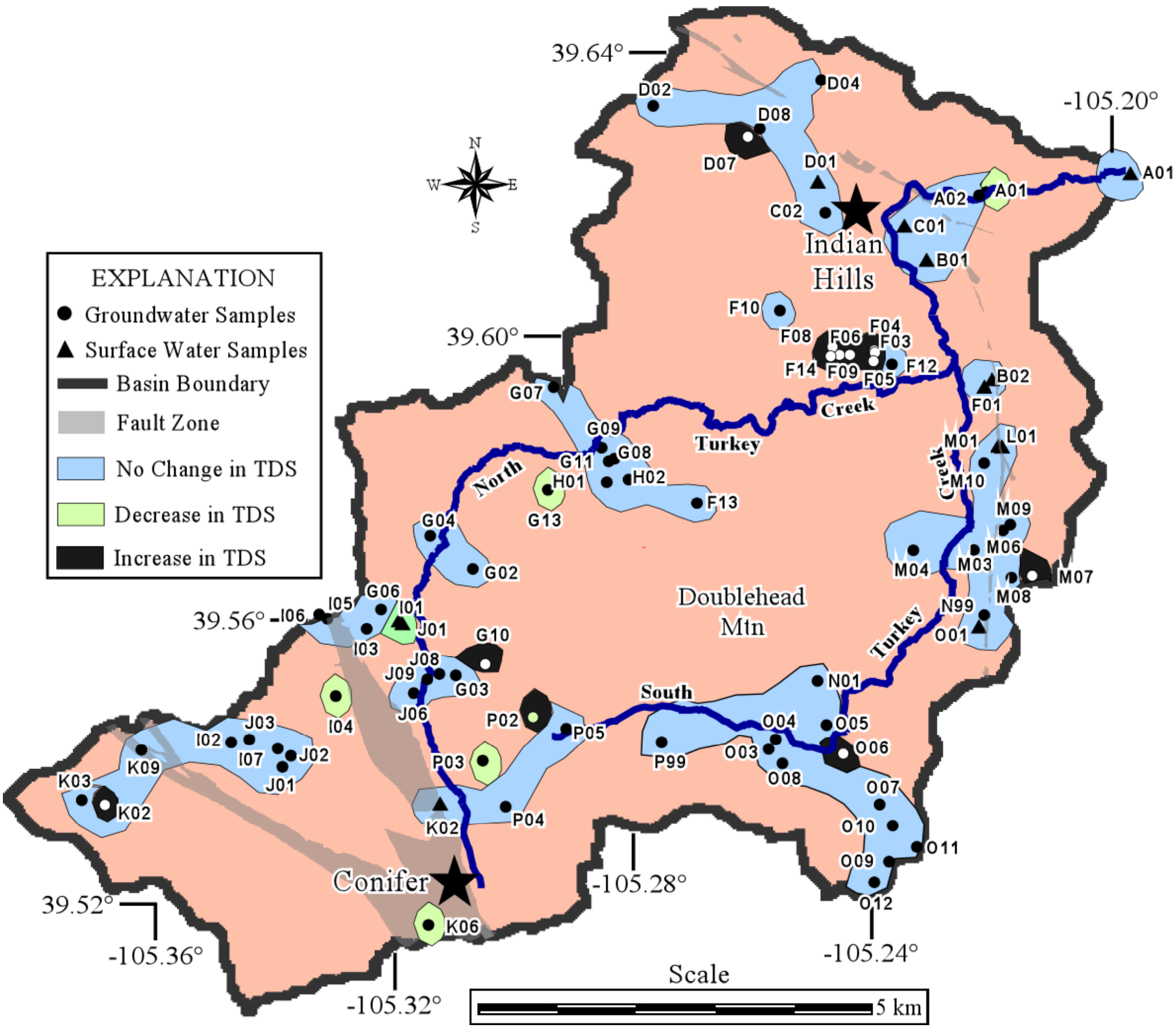

Figure 12. Locations of samples that increase, decrease, or exhibit no change in TDS between spring-runoff and baseflow data sets.

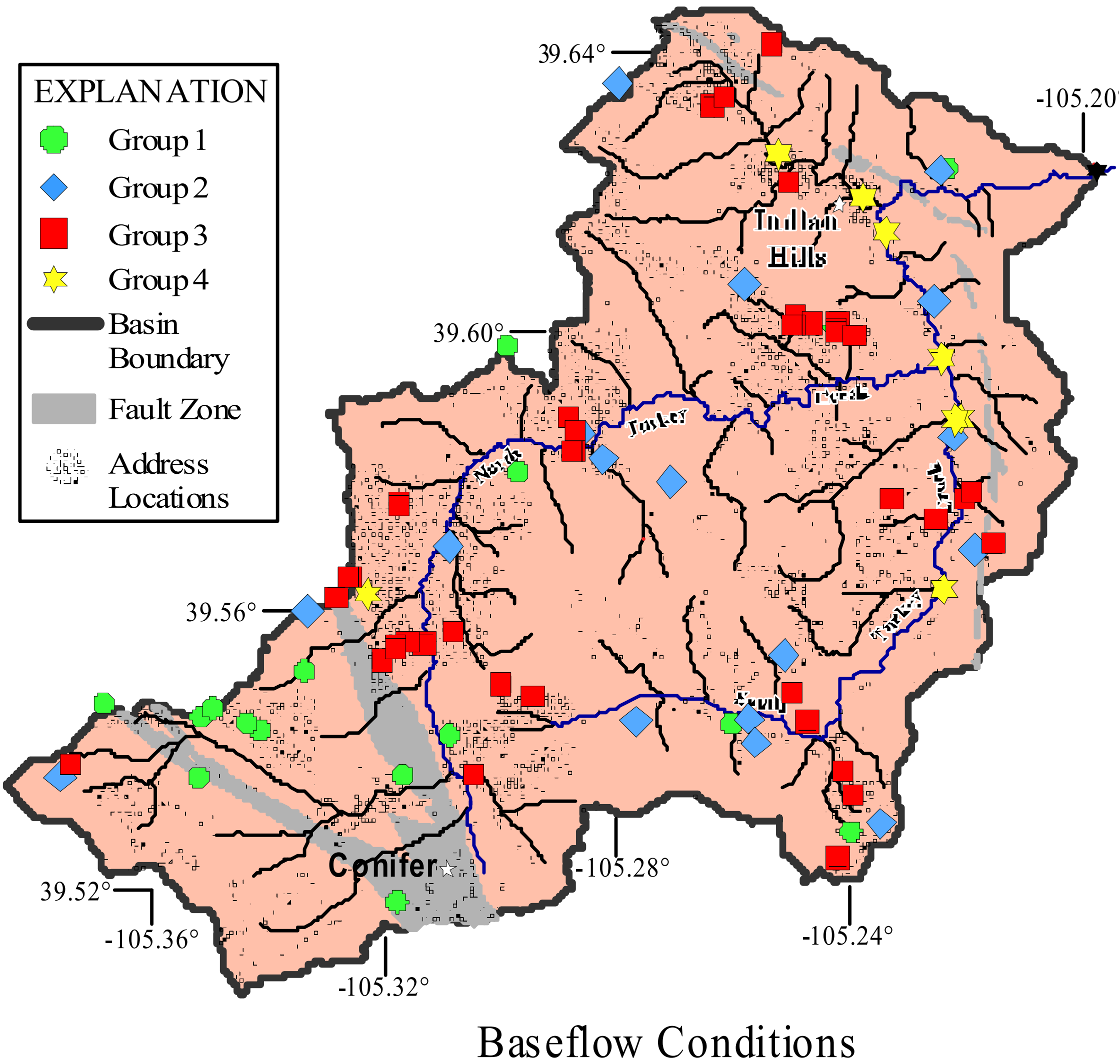

Figure 13. Locations of samples based on HCA grouping for baseflow sample set.

CLOSING

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Evaluation of front-range aquifers is only beginning and this project will continue to refine conceptual models and limit the possible range of interpretation through data mining, data collection, and analysis. Current analysis of the available data reveals:

- Water bearing fracture frequency is fairly uniform among rock types (~0.01 water-bearing fractures per foot) but fault zones and coarse granitic rocks have higher yields per fracture, thus are likely to have larger apertures and/or better connectivity.
- Fracture frequency (and yield) is uniform between 100 and 700 feet below ground surface.
- Well yields are higher in the fault zones and coarse granite, which occupy limited area in the upper portion of the basin.

- Depth to water averages less than 100 ft (30m).
- Water levels in wells mimic the topography, with coincident surface and ground-water divides.
- Water levels are responsive to spring recharge and generally exhibit a recession each water year.
- Precipitation is on the order of 20 in/yr (508 mm/yr) while evapotranspiration is on the order of 18 in/yr (457 mm/yr). Both are variable and known from a short period of limited spatial distribution, and thus introduce much uncertainty in the water budget.
- Water levels are declining.
- Storage in the basin is poorly characterized, but appears to be on the order of 2% in shallow zones.
- Volume of annual recharge is uncertain but is currently estimated on the order of an inch per year, with 75% pumped, but only 7% consumed because of ISDS recharge.
- Estimates of recharge are uncertain due to the short period of record and limited spatial distribution, consequently the estimate may be somewhat more or substantially less.
- The uncertainty associated with the water budget renders assessment of the sustainable population difficult.
- Surface water chemistry appears to have been adversely impacted by population growth during and after the 1970s.
- Ground-water chemistry has been impacted by anthropogenic effects that include high nitrate and chloride and lower pH, primarily in areas of high population density.
- Limited duration and spatial distribution of data prevents determination of whether the system has reached equilibrium concentrations.
- Ongoing studies will reduce current uncertainties.
- Hydrochemical data, water levels, and response of wells to recharge suggest an equivalent porous medium can represent the watershed for large-scale evaluations.
- Equivalent porous media models can be used to integrate the data, design further data collection and provide predictions of the hydrologic response to further development with ever decreasing uncertainty as additional data are accumulated.

Uncertainty associated with the Turkey Creek Basin water budget is large, making it difficult to determine the population that can be reasonably supported in the basin. Short-term records can be misleading, and must be used with caution. Water quality has been impacted by development, but the limited period of record prevents us from knowing whether concentrations have reached a steady condition or are reflecting only the beginning of a long-term increase. Continued collection of hydrologic records and assessment modeling is necessary to reduce uncertainty.

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APPENDIX: WEB SITES RELEVANT TO THIS PAPER

CSM, Turkey Creek Project http://www.mines.edu/~epoeter/research/TC/index.shtml

CoCoRaHs Rain and Hail Study http://ccc.atmos.colostate.edu/~hail/index.php

GROUND WATER CHARACTERIZATION OF THE BLUE RIVER WATERSHED, COLORADO TO ASSESS THE POTENTIAL IMPACTS OF ANTHROPOGENIC POLLUTANTS

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KeyTerms: Summit County, hydrology, aquifer, ground water, water quality, wastewater

ABSTRACT

Watershed-scale ground water characterization can be a challenge due to the scarcity and inherent uncertainties associated with publicly available data, coupled with the high costs of additional field scale investigations. In the Blue River Watershed of Summit County Colorado, we have combined limited supplemental fieldwork with publicly available data to evaluate the ground water hydrology and potential anthropogenic effects on water quality. This evaluation was focused on two locations in the Blue River Watershed: (1) Frisco Terrace, located near Dillon Reservoir; and (2) Blue River Estates, located along a tributary of the upper reaches of Blue River. The public data was obtained from water-well logs available from the Colorado State Engineer's Office (CSEO), and one year of surface water quality data collected by the USGS. Site-specific data included soil analyses from the borings of four monitoring wells, one year of hydraulic head measurements, monthly water-quality analyses from these wells, and one year of monthly stream-water quality data. Based on the CSEO records, fundamental properties of the aquifer were estimated and compared favorably to information obtained from the monitoring wells. Hydraulic conductivity (K) estimated from driller's pump-test data reported in the well logs are on the order of 10^{-2} to 10^{-3} cm/sec. These values compare favorably with K values estimated from particle analysis of soils collected during drilling (~ 10^{-2} cm/s). These K values suggest relatively fast chemical transport times. During 2002 (a dry year), the hydraulic head of the ground water at a study area (Blue River Estates development) in the upstream portion of the watershed was clearly lower than the nearby streams, indicating the streams are recharging the ground water. Pollution of nearby stream water from onsite wastewater systems (OWS) is unlikely in this scenario. However, the ground water appears to be discharging to wetlands located further downstream on the Blue River. At the study area near the mouth of the watershed (Frisco Terrace development), it is not clear whether ground water is recharging the streams based on physical data alone. A total of 76 surface and ground water samples were taken along the Blue River drainage between September 2001 and August 2002. The samples were clustered into six groups through chemical fingerprint analyses, each set with distinct chemical signatures due to differences in sulfate, chloride and nitrate concentrations. The water quality in the Blue River drainage varies in a predictable and systematic fashion. Surface water samples from the head of the Blue River, Pennsylvania Creek and Swan River show chemical

signatures of the natural water in mountain drainages. No anthropogenic impacts are noted in the streams at this location, supporting the contention that wastewater constituents are not entering the stream via a ground-water pathway. At the Frisco Terrace study area, waters exhibit elevated SO_4 possibly associated with mine tailings. In addition, the surface water in this area shows an anthropogenic signature that consists of elevated Cl and NO_3. Three of the four monitoring wells installed in the watershed have chemical signatures that are essentially identical to the local surface water systems. This implies that the ground water and surface water at those locations are clearly connected. However, one monitoring well shows a distinct chemistry with lower TDS and SO_4, but higher Cl and NO_3, indicating a lack of connection with the surface water system, and potential anthropogenic impacts.

INTRODUCTION

On-site wastewater treatment systems (OWS) are used for wastewater disposal supply for approximately 25% of the United States population, and the percentage is growing. Thus, it is important to consider the cumulative effect of many systems on water quality in sensitive or rapidly developing areas. The Blue River Watershed, a mountain watershed in Summit County Colorado (Figure 1), is an example of such a system. The largest water storage facility in the Denver Water System (Lake Dillon) is fed by the Blue River, which discharges to the Reservoir near Frisco. This watershed is currently being evaluated by Colorado School of Mines, to develop and test a methodology for assessing the water quality impacts of decentralized wastewater systems, including individual and cumulative effects on water quality.

The overall effort includes surface-water quality and flow assessments for the Blue River and selected tributaries, and the lower part of Ten Mile Creek (which empties into Lake Dillon via Frisco Bay). Ground water characterization, watershed-scale modeling and pollutant-transport evaluations are also being conducted to enhance the understanding of the transport/fate of microbes and chemicals from OWS, and provide a template for studying watershed-scale effects of OWS.

The specific purposes of this paper are to:

1. Assess the general character of the aquifer used by households in the Blue River watershed (e.g., depth to ground water, sedimentary versus fractured-bedrock aquifer, degree of heterogeneity).
2. Characterize the general hydraulic properties of the aquifer useful for estimating pollutant transport (e.g., hydraulic conductivity, bulk density, porosity, etc.,).
3. Determine if aquifer conditions are generally conducive to ground water/surface-water interactions that could allow transport of OWS to streams and thus to Lake Dillon.
4. Identify if OWS in the Blue River watershed are significant sources of nitrogen (N) and phosphorus (P) inputs in this system, aiding in the eutrophication of Lake Dillon.

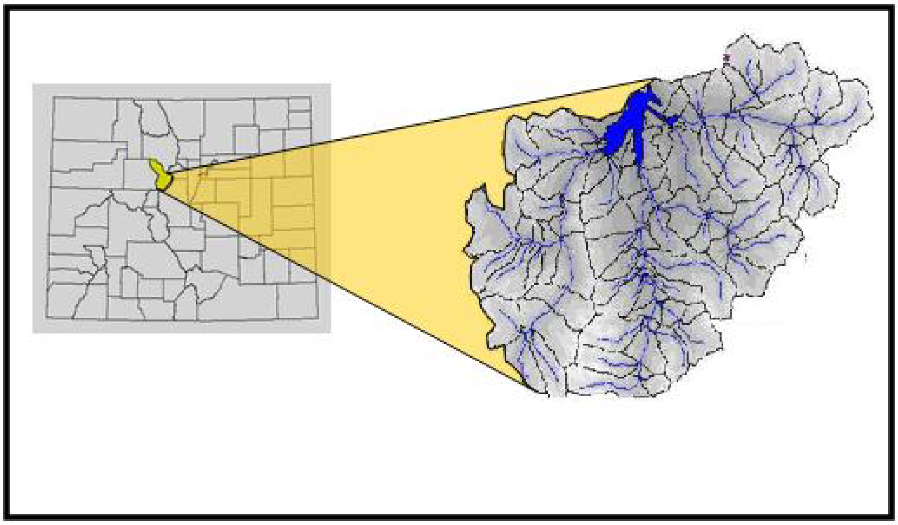


Figure 1. Location of Southern Summit County Colorado

These items are important for several reasons. First, it is often not clear whether aquifer systems in these mountain watersheds are comprised primarily of sediments (i.e., a typical heterogeneous porous-media aquifer) or of fractured bedrock. This distinction has serious implications on OWS pollutant transport, and the hydraulic connection between OWS and the drinking-water source (aquifer) and the surface-water system. For example, drinking water wells located in another relatively close proximity mountain watershed, Turkey Creek Watershed, Jefferson County CO, are drilled primarily into fractured bedrock. It is unclear in this system whether OWS effluent discharges directly to the aquifer, or if effluent is collecting on the top of bedrock, with some infiltration to the aquifer and some effluent flowing to creeks in the near-surface water system. In a sedimentary aquifer, the depth of porous materials between OWS effluent and the top of the water table is crucial to estimate, because the soils serve as a treatment media for OWS pollutants.

Identification of the general hydraulic properties of the aquifer is necessary to better understand the ground water flow and pollutant transport characteristics, allowing for the construction of watershed-scale ground water and pollutant transport models. Finally, in watersheds where the potential pollution of surface-water bodies from OWS effluent is of concern, identifying the potential for ground and surface water interaction is very important. In the Blue River watershed, the eutrophication of Lake Dillon with nutrients such as nitrogen (N) and phosphorus (P) is an ongoing problem.   In particular, other researchers have identified OWS as a potential cause of P pollution (Lewis et al., 1984). These issues are addressed in this preliminary characterization of the ground water system in the Blue River watershed.

STUDY AREA

Summit County, Colorado is located approximately 65 mi west of Denver in the central Rocky Mountain Region of the state. The county's boundaries are the Eisenhower Tunnel to the east, the top of Vail Pass to the west, Hoosier Pass to the south, and Green Mountain Reservoir to the north. The highways that serve as major access to the county are I-70 from the east and west, and State Highway 9 from the north and south. Summit County was the fastest growing county in the country from 1970 to 1980 with an increase in population of 232%. Summit County's current population is approximately 24,000 and it remains one of the fastest growing counties in Colorado. The county has four primary population centers: Breckenridge, which is the oldest town and serves as the county seat, Dillon, Frisco, and Silverthorne. The majority of residents live in unincorporated areas. Summit County has an average annual snowfall of 159.4 inches, and is the home of four major ski resorts: Arapahoe Basin, Breckenridge, Copper Mountain, and Keystone. Elevation ranges from 7,947 feet to 14,270 ft (2422 to 4349 m) above mean sea level (amsl). (Summit County Tourism Web Page, www.co.summit.co.us). Figure 2 illustrates the major sub-watersheds in the Blue River watershed. Our focus is on the Blue River watershed south of Lake Dillon, and a portion of Ten Mile Creek near the mouth where it empties into Lake Dillon.

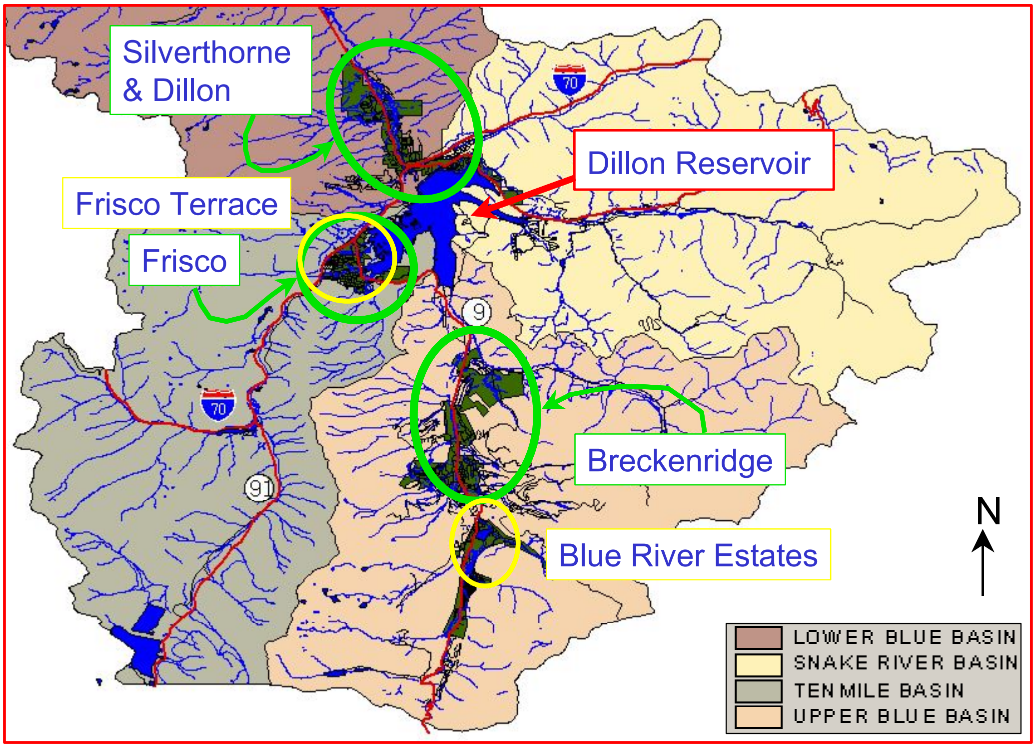


Figure 2. Watersheds in Summit County Colorado (Lemonds 2003)

Monitoring wells were emplaced in two areas (Figure 3): Frisco Terrace (FT), a community of 1,263 acres (5.1 km^2) located at the edge of the Western Arm of the Dillon Reservoir (adjacent to I-70) at an elevation of approximately 9040 ft msl (2755 meters msl); and Blue River Estates (BRE), a residential development comprised of 2200 acres (8.9 km^2) of unincorporated land, located up gradient on the Blue River (south) from Breckenridge. BRE exhibits elevations changes that range from near the continental divide (10,277 ft msl; 3132 m msl), down to the flood plain at the confluence of Pennsylvania Creek (Penn Creek) and the Blue River (9979 ft msl; 3042 m msl). Details on the monitoring wells are presented later in this paper.

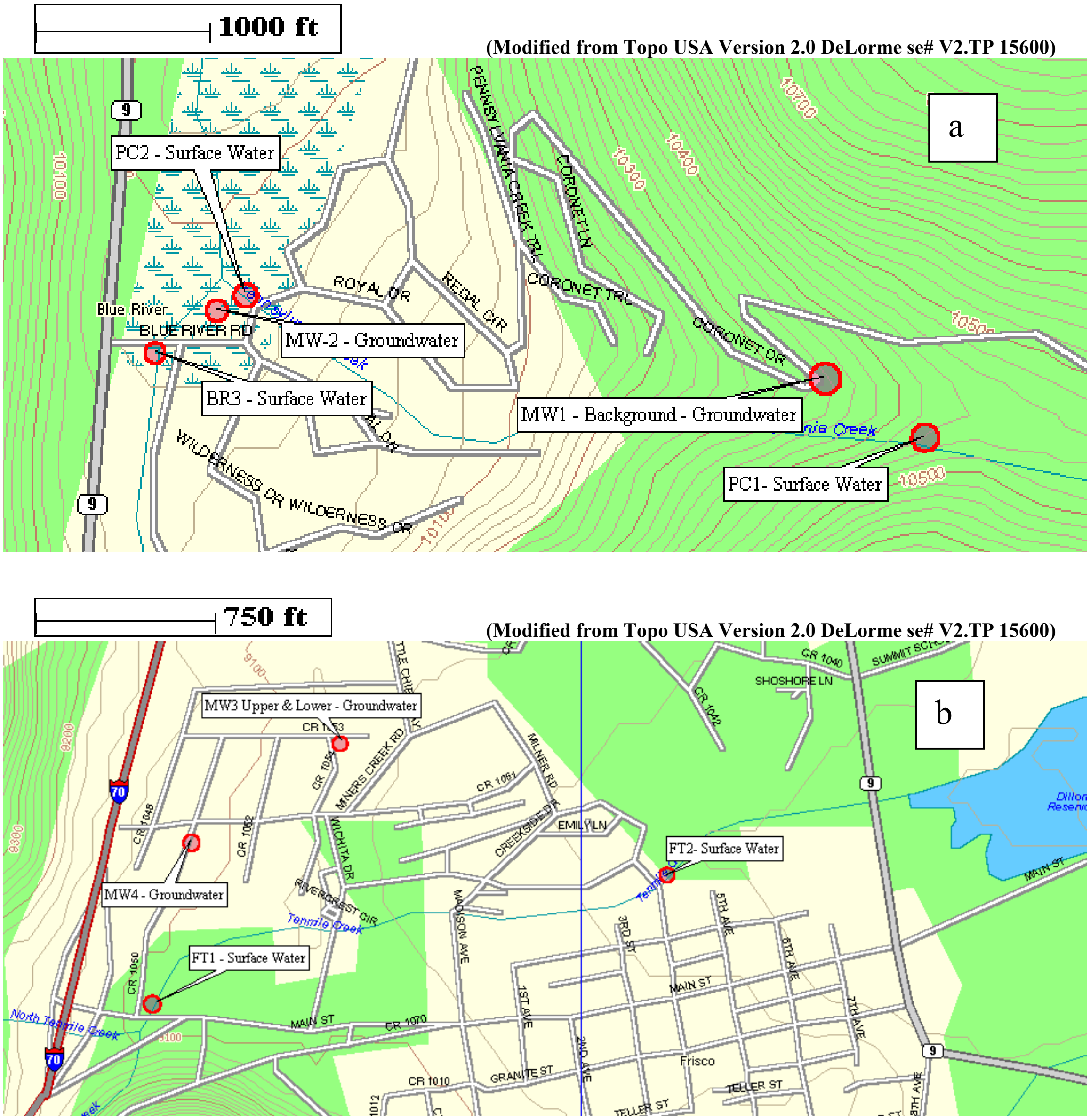


Figure 3. Study Areas at (a) Blue River Estates and (b) Frisco Terrace

DATA SOURCES

Analyses of Well Logs from the Colorado State Engineer's Office

One low-cost, preliminary method for watershed-scale ground water characterization is the analyses of the available Colorado State Engineer's Office (CSEO) well logs. The CSEO maintains free databases that are computer-searchable by many keywords. Additional information is often available on the hard copies of these well logs. We have reviewed more than 5000 well logs for this and other watershed studies.

Table 1 lists some of the useful information typically found on a well log, along with a qualitative assessment about the frequency that each type of information is recorded. Information in these logs is compiled at the time of well completion, usually by the driller. Thus, the data can span over 50 years or more, depending on the time period for which development has occurred in a particular area. While this information is not always considered to be highly accurate, it provides an excellent first-estimate of the ground water hydrology. In many cases, this is the only data on the ground water system that is available for a watershed-scale study.

Table 1. Available Information from Typical CSEO Drilling Logs

Parameters Potentially Available on Colorado State Engineer Drilling Log Records	**Degree of Frequency that Information is Available on Drilling Log Records**
Drilling Methods (i.e. rig type)	Always
Quarter Quarter Section Location	Frequently
Specific GIS or Survey Location Information	Rarely
Well Permit or ID number	Always
Well Owner at Time of Permit Issue	Often
Well Owner at Current Time	Occasionally
Total Well Depth	Always
Well Drilling Completion Date	Often
Casing Sizes and Depths	Occasionally
Perforated Casing Size and Depth	Occasionally
Grouting Materials	Occasionally
Grouting Intervals	Occasionally
Pump Test Date	Often
Static Water Level Prior to Pump Test	Often
Length of Pump Test	Rarely
Sustained Yield	Often
Final Water Level after Pumping	Rarely
Hole Diameter	Often
Depth to Rock (if applicable)	Often
General Lithology Descriptions (i.e. fill, sand, rock, soil)	Often
Specific Lithologic Descriptions (i.e. med. gr. Brown quartz sand, limestone, granite, coal veins, etc.	Rarely

Always = recorded 90%-100% of the time
Often = recorded 65%-90% of the time
Occasionally = recorded 40%-65% of the time
Rarely = recorded <40% of the time

Two hundred seventy eight (278) individual well logs for residences within the BRE area, and 87 logs for wells in the FT area were reviewed for a description of the subsurface lithology, aquifer material, and aquifer location. Initial evaluation of the well logs for BRE and FT verified that nearly all the drinking water wells were set in porous unconsolidated sedimentary material rather than in fractured bedrock. Nearly all of the drinking water wells are installed in medium to course-grained sand to depths averaging 80 ft below ground surface (bgs). The majority of the development in Summit County, has been within the glacially carved valleys, which are filled with glacially derived sandy alluvium. No discernable watershed-scale confining unit was observed or logged in the CSEO well logs to depths up to 200 ft (61 m) bgs. Thus, the ground water and surface water at this site has potential to be connected. The septic systems and drinking water wells in our study areas are also installed within this glacially derived course grained gravelly sand and boulders, so it is feasible that OWS pollutants could enter streams via a ground-water pathway.

Frisco Terrace's average well depth is 83 ft (25 m) bgs (geometric mean is 67 ft or 20.5 m) with a standard deviation of 74 ft (22.5 m). The range is 22 to 310 ft (7 - 95 m). The average depth to water in FT is 38 ft (11.5 m) bgs (geometric mean is 29 ft. or 8.9 m) with a standard deviation of 35 ft (10.5 m). The range is 9 ft to 160 ft (3 - 49 m). The average depth of wells in the BRE Area is 94 ft (28.5 m) (geometric mean is 76 ft or 23 m) with a standard deviation of 60 ft (18 m). The range is 27.6 to 520 ft (8.4 to 158.5 m). The average depth to water in BRE is 49 ft (15 m) (geometric mean is 37 ft or 11 m) with a standard deviation of 36 ft (11 m). The range is 4 ft to 170 ft (1.3 to 52 m). This information is useful, for example, to plan drilling depths for monitoring wells, or for newer drinking-water wells.

Figure 4 illustrates that the depth to water has steadily declined as wells are installed. Interesting, the trends for two sites are very similar, although many more wells have been installed at BRE than FT. Figure 4a and 4b indicate that water levels have lowered and over the last 41 years as the number of wells installed as increased. Figure 4c shows the water level trend versus cumulative number of wells installed. These trends reflect the rapid population growth in the study areas. Interestingly, the depths to water have appeared to reach a relatively steady level in the past 10 years. This may occur because of increased stream recharge to the aquifers. Unfortunately, stream flow data does not exist below the development to evaluate this possibility. A gaging station exists downstream of BRE, but it is located at the outfall of the man-made reservoir at Goose Pasture Tarn, so a direct comparison cannot be made. The relatively constant water levels may also exist because the ground-water system has reached steady state with respect to pumping of wells, recharge from precipitation and septic tank effluent, and influx of surface water to the system. The close proximity of Lake Dillon to the FT subdivision, also likely helps regulate the water levels in this area by acting as a source when needed to equilibrate the potentiometric surface of the water table.

At BRE, the number of wells installed has increased significantly while the water table levels have actually increased slightly in recent years. One potential reason for this is that the homes built in the BRE area in the past 20 years are generally built on larger lots, farther away from other houses and from the stream system. Thus, the influence of growth on the water table may be mitigated.

Estimating Hydraulic Conductivity from the Specific Capacity Information on Well Logs

Nearly all CSEO well logs contain some form of pump-test information. To estimate hydraulic conductivity (or transmissivity) two pieces of information are needed: drawdown and pumping rate. Still, some basic assumptions were made. The first is that the recorded pumping rate reflects a sustained rate of recharge from the formation, after water is removed from storage in the well bore itself. The second assumption is that when drawdown is recorded as "all" or "total" in the well logs, the total drawdown can be estimated by total borehole depth minus the reported depth to water. The assumption of negligible well-bore storage was deemed sufficient

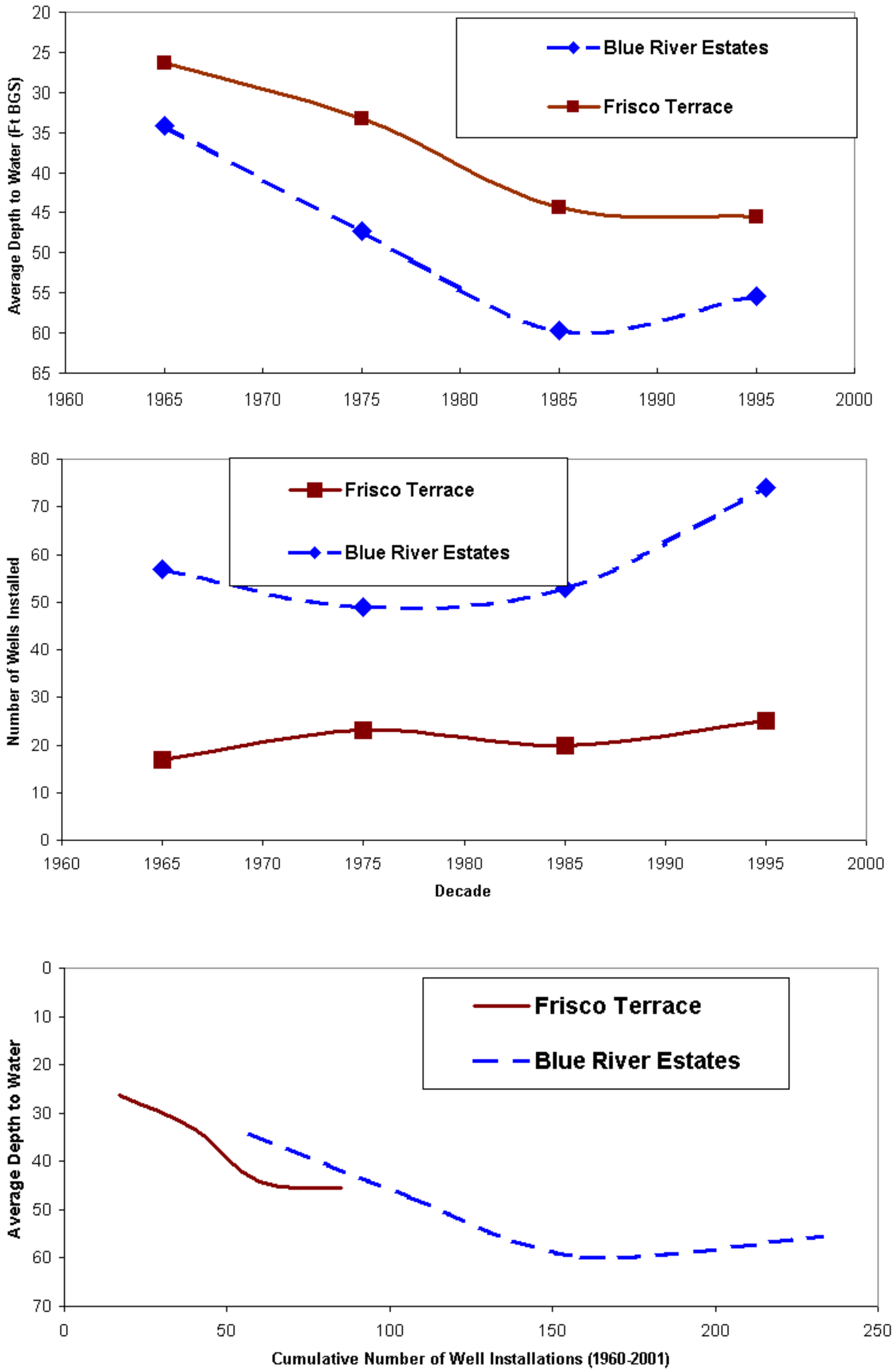


Figure 4. CSEO data for (a) Average water level by decade since 1960, (b) Number of wells installed by decade, (c) Cumulative number of wells installed vs. depth to water. BGS = below ground surface.

if the time required to empty the well for the reported pumping rate (assuming no recharge from the formation) was less than 20% of the pump test duration.

Thus aquifer transmissivity (T) may be estimated by as:

$$T = 1140\ (SY / \Delta h)^{0.67} \qquad (1)$$

where T = Transmissivity in square feet per day, SY = sustained yield in gpm, Δh = drawdown in feet (Razack and Huntley, 1991). This equation is empirical and was developed to estimate transmissivity from specific capacity data in a large alluvial aquifer. Other more detailed expressions are available (Fetter, 2001), but these require estimates of the specific yield, which are not available. Equation 1 allows a first estimate of T for BRE and FT (Table 2). No pump tests information is available in either of these two study areas for comparison to the estimates provided in Table 2.

Table 2. Transmissivity Estimated from Equation 1 and CSEO Well Log Information

Site Location	**Average Transmissivity**	**Std. Dev. of Transmissivity**	**Geometric Mean of Transmissivity**	**Data Range**
Blue River Estates (BRE)	773 ft^2/day (8.3 cm^2/sec)	497 ft^2/day (5.3 cm^2/sec)	647 ft^2/day (6.9 cm^2/sec)	153-2106 ft^2/day (1.6 – 22.6 cm^2/sec)
Frisco Terrace (FT)	585 ft^2/day (6.3 cm^2/sec)	395 ft^2/day (4.2 cm^2/sec)	416 ft^2/day (4.5 cm^2/sec)	41–1403 ft^2/day (0.44 – 15.1 cm^2/sec)

Hydraulic conductivity (K) may be estimated using equation 2 below. Aquifer thickness, b, must be assumed. For these study areas, we assumed two cases for aquifer thickness. A thickness of 50' is a reasonable minimum aquifer thickness, which approximately corresponds to the distance between the average water level and the average well depth. For the other end member, the maximum aquifer thickness is taken to be the average depth to the bedrock (based on well logs and geologic map information). The results are shown in Table 3.

$$K = T / b \qquad (2)$$

Table 3. Hydraulic Conductivity Estimates from CSEO Well Log Information

Site Location	**Depth of aquifer thickness**	**K (cm/sec) from Average T**	**K (cm/sec) from Geometric Mean of T**
Blue River Estates	50' (1524cm)	5.4×10^{-3} cm/s	4.6×10^{-3} cm/s
Blue River Estates	185' (5639 cm)	1.5×10^{-3} cm/s	1.2×10^{-3} cm/s
Frisco Terrace	50' (1524 cm)	4.1×10^{-3} cm/s	2.9×10^{-3} cm/s
Frisco Terrace	165' (5029 cm)	1.3×10^{-3} cm/s	8.9×10^{-4} cm/s

The estimated K values are relatively well constrained to within a half order of magnitude. The BRE values correspond well to slug tests performed in the upper and lower screens at MW-1 and MW-2 (discussed below), which yielded K values between 1 x 10^{-3} cm/sec and 8 x 10^{-3} cm/sec. These well-log estimates appear to provide a significant improvement over an estimate of K based strictly on porous-media type, whereby K can vary over at least 4 orders of magnitude (Fetter, 2001). K values in the 10^{-3} range are conducive to relatively fast transport times. Thus, pollution of streams from known or suspected areas of contaminated ground water, or spreading of pollution within ground water systems, is a concern.

Installation of Monitoring Points in the Study Areas

Four monitoring wells constructed of 1-in diameter PVC were installed to collect site-specific ground-water information. A small mobile Auger drilling apparatus was used to create the boreholes. The four borings ranged in depth from 30 to 40 ft below ground surface. Some of the borings had two PVC monitoring wells installed in them (shallow and deep). The monitoring points are generally 1-ft long screen sections, located near the bottom of each well.

These well locations are shown on Figure 3 and include:

- A "background site" located at the edge of the BRE development (and at a higher elevation) that we assume is unimpacted by development (MW-1).
    - This is a nested well pair, but the upper well is usually dry;

- A location within BRE near the confluence of Blue River and Pennsylvania Creek (MW-2).
    - This well is also a nested well pair, but the upper well is generally dry.

- Two locations in Frisco Terrance (MW-3 and MW-4), which represent a highly populated area along Ten Mile Creek near the exit of the stream into Lake Dillon.
    - MW3 shallow and deep nested wells. Thus MW3 water-level measurements and ground water samples are designated MW-3U (upper) and MW-3L (lower) to differentiate between the two monitoring depths at this location.
    - MW4 is a single well.

Figure 5 illustrates the water levels in each monitoring point collected from February 2002 to January 2003. The water levels in these monitoring wells are considerably shallower than the water levels obtained from CSEO logs for the 1990s. This is probably due to a combination of two reasons: (1) the monitoring wells were installed near the streams while the CSEO logs are averages for the entire developments; (2) drillers record the water table depth before it has completely rebounded, which is likely to yield larger than actual depths to ground water. Note that the water levels are relatively stable in all the wells with the exception of MW2, which is located approximately 200 feet from Pennsylvania creek within the flood plain between Pennsylvania Creek and the Blue River. In MW-2, the water levels are significantly lower in the winter months. The reason for this behavior is not fully understood; however, the times when the observed water levels drop coincide to the times of complete freeze of Pennsylvania Creek. Thus, the change in water levels could be due to cessation of ground-water recharge from the stream at this location because the stream is frozen. This would imply that Pennsylvania Creek was a losing stream during the period that water levels were collected. Furthermore, it suggests a strong connection between ground water and surface water at this location.

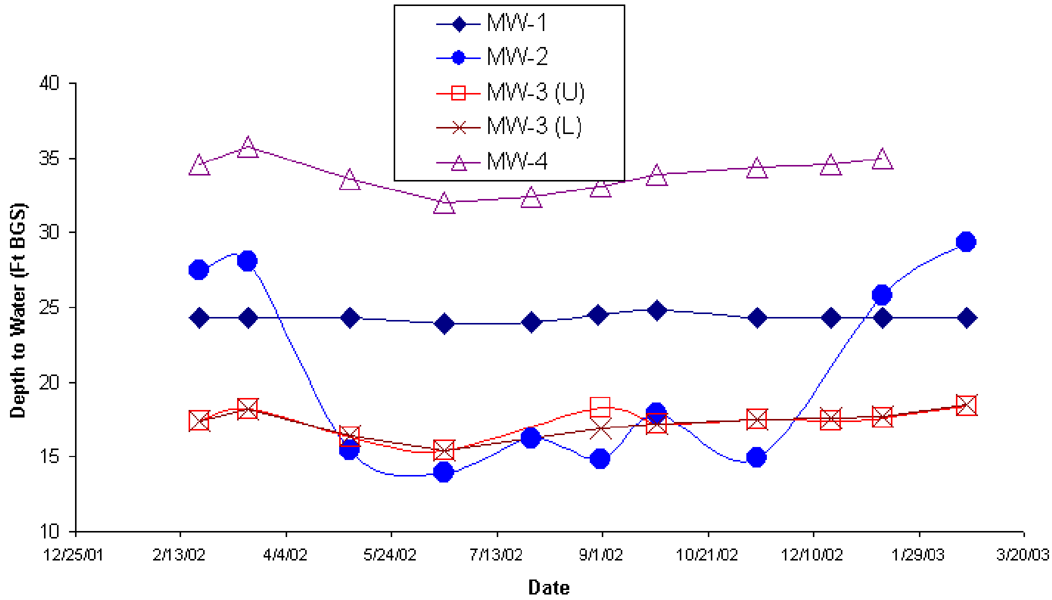
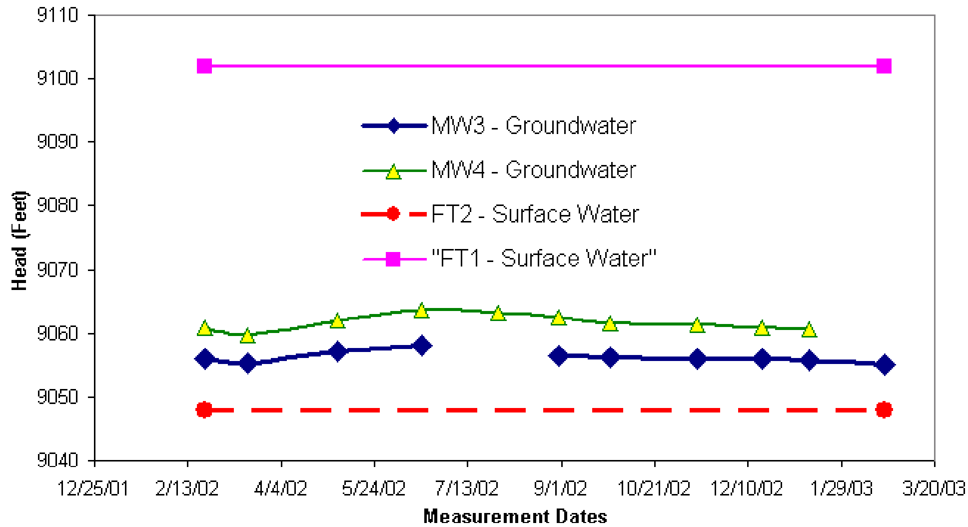


Figure 5. Monitoring Well Water Levels (Dec. 2001-Jan. 2003).

Figure 6 shows hydraulic head in each well compared to the stream elevations (Figure 6a and 6b). Hydraulic head values were computed for ground water from GPS measured elevations at the base of the well, measured stick-up heights of the monitoring-well casing tops, and measured depths to ground water from the casing tops. The error in this measurement is about ± 3 ft (1 m) due to the uncertainty of the GPS measurement. The hydraulic head of the streams are measured from GPS elevations of the stream surface near the beginning of the study. The stream levels are known over time, but vary by less than about a foot during the study period. Thus, the error in these values is about ± 4 ft (~1.3 m). However, we believe the relative error between locations, all collected with the same GPS unit, is probably more accurate. MW-3 is shown as only one value because the water levels and tops of casings were essentially identical (recall Figure 5). Stream elevations are shown to be constant for simplicity.

The head data is useful to assess potential contamination of streams from OWS effluent, which discharges to the subsurface. Thus, the pathway for OWS pollutants to enter the stream system is via ground water discharging into the stream. If the stream consistently feeds the ground water system, then OWS pollution of the nearby streams is not likely. In the Frisco Terrace focus area, the flow patterns are complex because of the nearby lake, the close proximity of steep hills and mountains that facilitate local recharge, and city-setting where discontinuous pavement can also cause local recharge. Thus, additional monitoring wells would be required to ascertain flow paths. It is not clear whether the ground water is discharging to the stream or vice-versa. However, we can make a realistic assessment based on the data. The monitoring wells are in between stream-water stations FT-1 (upstream) and FT-2 (downstream), although much closer to FT-1 (Figure 3). The head at FT-1 is significantly higher than in the MW. In addition, the head in MW-4 (closer to Ten-mile Creek) is higher than the head in MW-3 (farther from the creek). This evidence suggests that the stream may be discharging to ground water, which indicates that OWS pollution of the streams is not likely. The ultimate discharge of the ground water system is the Frisco Bay at the inlet to Lake Dillon. However, under the current monitoring stream samples is not a useful method to assess OWS input from Frisco Terrace development into Lake Dillon.



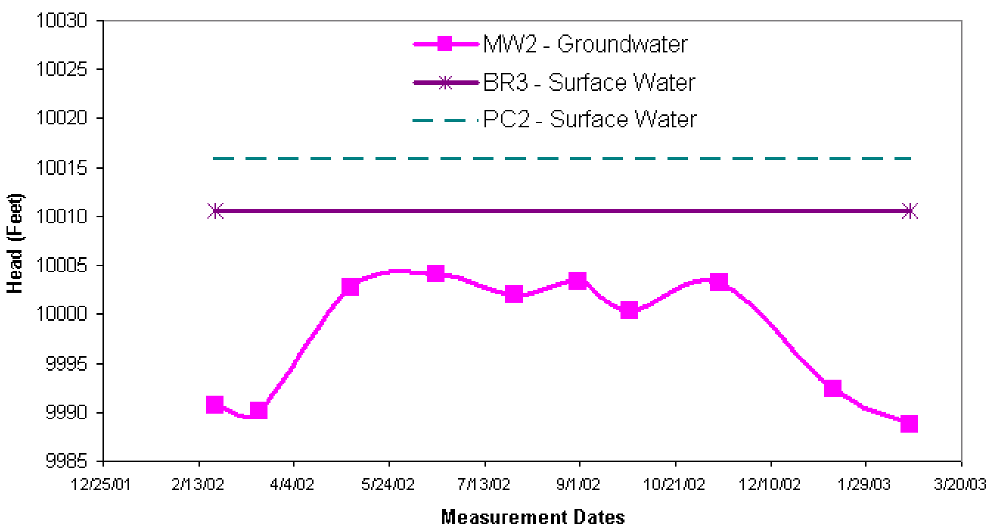


Figure 6. Hydraulic head for ground water and surface water at (top) Frisco Terrace, (bottom) Blue River Estates. Elevation reference is mean sea level. Average head at MW-1 is 10291 ft (not shown).

In the Blue River Estates focus area, the head of the MW-1 ground water (not shown, but listed in Figure 6 caption) is at a much higher elevation than MW-2, and the head in MW-2 is somewhat higher than the elevations of the nearby streams (Figure 6). Thus, ground water is currently flowing from higher elevations to lower elevations towards, and likely underneath, the Blue River and Pennsylvania Creek. This conclusion is consistent with the discussion of the frozen-stream recharge effects on MW-2 water levels (Figure 5). The probable discharge point for this groundwater (based on flow-direction analysis) is the wetlands in Goose Pasture Tarn several hundred meters south of MW-2. This is sensible because, in this geomorphic setting, ground-water discharge is the most likely source of a wetland. Thus, under current hydrologic conditions, wastewater-pollutants from OWS systems are not likely to enter the nearby streams, but rather would enter the Blue River system via the Tarn.

For both of the above scenarios, however, it is important to note that this data is based on monitoring conducted during a drought year; therefore, especially in the cases of MW3 and MW4, wetter years may result in ground water discharging into nearby streams. Recently collected data (that has not yet been quality assured) during 2003 (a wetter than average year) indicate the same trends as above. However, it is not known how long it would take for "typical" hydrologic conditions to return after a several year drought as occurred from 1999-2002. Nonetheless, for both scenarios above, potential ground water flow paths into surface-water bodies are longer than originally hypothesized, which would result in larger attenuation of pollutants due to soil sorption and other biochemical reactions.

The data above is also useful to estimate pollutant transport times. The BRE development is used as an example here. The ground water velocities across the Blue River Estates development (the source of potential OWS pollutants) are relatively fast at the BRE site. The average hydraulic head difference between MW-1 (just on the up gradient side of the development) and MW-2 (down gradient of the development) is about 280 ft. The approximate distance between these along a flow path perpendicular to the hydraulic gradient is 2900 ft. This yields a large hydraulic gradient of about 0.1. Assuming a porosity of 0.3 and applying Darcy's Law gives a ground-water velocity on the order of 10^{-3} cm/sec (~1000 ft/yr). Thus it would take ground water about 1.5 years to travel from the center of the BRE development past MW-2. The gradient from MW-2 area to the wetlands is significantly less (about 0.01). This yields a travel time of 4 years. Total travel time is therefore about 6 years. However, contaminants may travel slower than the ground water due to sorption to soil or other reactions. For example, phosphorous (P) retardation factors are in the range of 10 to 100 for most soils (Kirkland, 2001; McCray et al., 2003). The best-case scenario would thus yield a P travel time of more than 400 years. Clearly, to get an accurate estimate of pollutant travel times, more monitoring wells must be installed to better ascertain the hydraulic gradient and to obtain a better statistical representation of K, and soils must be analyzed for site-specific sorption parameters. Travel times at the FT study area would be significantly slower because of much smaller hydraulic gradients across the development, and from the development to the likely discharge point into Lake Dillon.

Soils Physical Data

Continuous soil samples were collected during the first 10 ft (~3 m) of boring, during well installations, and select samples were tested for soil-classification (% sand, % silt and % clay using the hydrometer method). Table 4 summarizes the results.

Table 4. Soil Classifications at Each Monitoring Point Location

Focus Area	Location	Depth (cm bgs*)	Depth (ft bgs*)	% Sand	% Silt	% Clay
Blue River Estates	**MW-1 (Background Well)**	75	2.5	60	27	13
		180	6	63	23	14
		330	11	81	12	7
Blue River Estates	**MW-2 (Blue River /Pennsylvania Creek Flood Plain)**	60	2	75	19	6
		180	6	73	19	8
		300	10	72	19	9
Frisco Terrace	**MW-3**	60	2	78	15	7
		180	6	80	14	6
		300	10	76	16	8
Frisco Terrace	**MW-4**	60	2	81	13	6
		180	6	88	7	5
		300	10	73	19	8

*bgs = below ground surface

The soils from MW-2, MW-3, and MW-4 all seem to have a similar size distribution. The materials are primarily sand (72-88%), with significant amounts of silt (generally 13-19%), with a relatively small amount of clay (less than 5-9%). The classifications at MW-1 (the background well located at the highest sampling point in the watershed) are also similar to those in the other wells for soils that are deeper than about 150 cm BGS. In the upper soil layers at MW-1, however, there is somewhat more silt and clay, but the majority of the soil is coarse (~60% sand) sand. For all the monitoring points, the average % sand is 75% (standard deviation 8%), the average silt value is 17% (standard deviation 5%) and the average clay composition value is 8% (standard deviation of 3%). Thus, the soil-particle analysis is consistent with the well-log information, albeit much more detailed.

In addition to the analysis reported in Table 4, a soil-sieve analysis was performed on selected soil samples from BRE and FT monitoring point boreholes. Sieve results are given in Table 5. The results are summarized by two effective grain-size diameters, D_{10} and D_{60}.

Table 5. Soil-Sieve Analysis for Samples from Monitoring Points

Sample ID and depth interval	**D_{10} (mm)**	**D_{60} (mm)**
MW4 (1.5-3.5'bgs) Frisco	0.20	6.0
MW4 (10-12'bgs) Frisco	0.40	7.0
MW2 (2-3.5'bgs) Pennsylvania Ck	0.15	0.4
MW2 (6-7'bgs) Pennsylvania Ck.	0.38	7.0
MW2 (11-12'bgs) Pennsylvania Ck	0.60	8.0

From the effective grain size diameter, represented by the D_{10} value, several different methods can be used to estimate hydraulic conductivity (K). These equations, 3a through 3c are judged valid for the range of D_{10} values measured. Details about these equations are provided in the text by Kresic (1997).

$K= (g/\nu)(6x10^{-4} \log (500/U))D_{10}^2$ Breyer Equation (3a)

$K = (g/\nu)(6x10^{-4}) (1 + 10 (n-0.26))D_{10}^2$ Modified Hazen Equation (3b)

$K = (g/\nu)(6x10^{-4}) n^3 (1-n)^2 D_{10}^2$ Kozeny Equation (3c)

where $g = 9.807$ m/s^2, ν is kinematic viscosity (1.14 x 10^{-6} m^2/s for ground water at 15 degrees C), n is porosity, $U = D_{60}/D_{10}$ is the uniformity coefficient, D_{10} is the sieve size passing 10% of the sample, D_{60} is the sieve size passing 60% of the sample.

Table 6 presents the geometric mean K values calculated for each of the three methods. These values are an order of magnitude larger than the ones values estimated using the engineer's well logs (Table 3), and the few slug test values reported earlier. However, this method also suggests that the K at BRE is somewhat higher.

Table 6. Hydraulic Conductivity Estimates from Grain-Size Analysis

Site Location	Breyer K (cm/sec)	Hazen K (cm/sec)	Kozeny K (cm/sec)
Blue River Estates	9.4 x 10^{-2}	7.6 x 10^{-2}	4.1 x 10^{-2}
Frisco Terrace	5.5 x 10^{-2}	5.8x 10^{-2}	3.1 x 10^{-2}

GROUND WATER CHEMISTRY

All the monitoring points installed by the Colorado School of Mines for purposes of generalized monitoring of this watershed, with the exception for MW4 (which is installed in FT), are two-level sampling wells (shallow and deep). These two-level systems are installed in the same borehole. Well depths average 20 ft bgs in the upper wells and 30 to 40 ft bgs in the lower wells. Water quality parameters were evaluated in the field, using a YSI Water Quality Checker that includes temperature, salinity, specific conductivity, total dissolved solids (TDS), pH, and dissolved oxygen. HACH laboratory analyses in the included: ammonia, chemical oxygen demand, pH, alkalinity, solids and dissolved solids. IC and ICP analyses provided information on cation and anion concentrations including: fluoride, chloride, nitrate, phosphate, sulfate, silver, aluminum, arsenic, boron, barium, beryllium, calcium, cadmium, cobalt, chromium, copper, iron, lithium, magnesium, manganese, molybdenum, nickel, phosphorus, potassium, scandium, selenium, sodium, strontium, sulfur, tin, titanium, vanadium and zinc.

Ground water pH values are shown in Figure 7. The highest values were detected in the background well, MW1 (between 7.8 and 8.5), and in MW2 (between 7.6 and 8.2) at the highest elevations. The pH soil values from the FT area, located near base level in the watershed are lower (between 6.5 and 7.5). Decreased pH levels in the Frisco Terrace area (drained by 10 Mile Creek) may be a reflection of historic mining activities in that area. While pH appears to trend higher in the winter months, a clear pattern of temporal variation is not apparent in this data set. The decrease in pH observed in February 2003 is likely an anomaly resulting from a data collection problem during field parameter collection, as it was uncommonly cold, and the pH meter was affected.

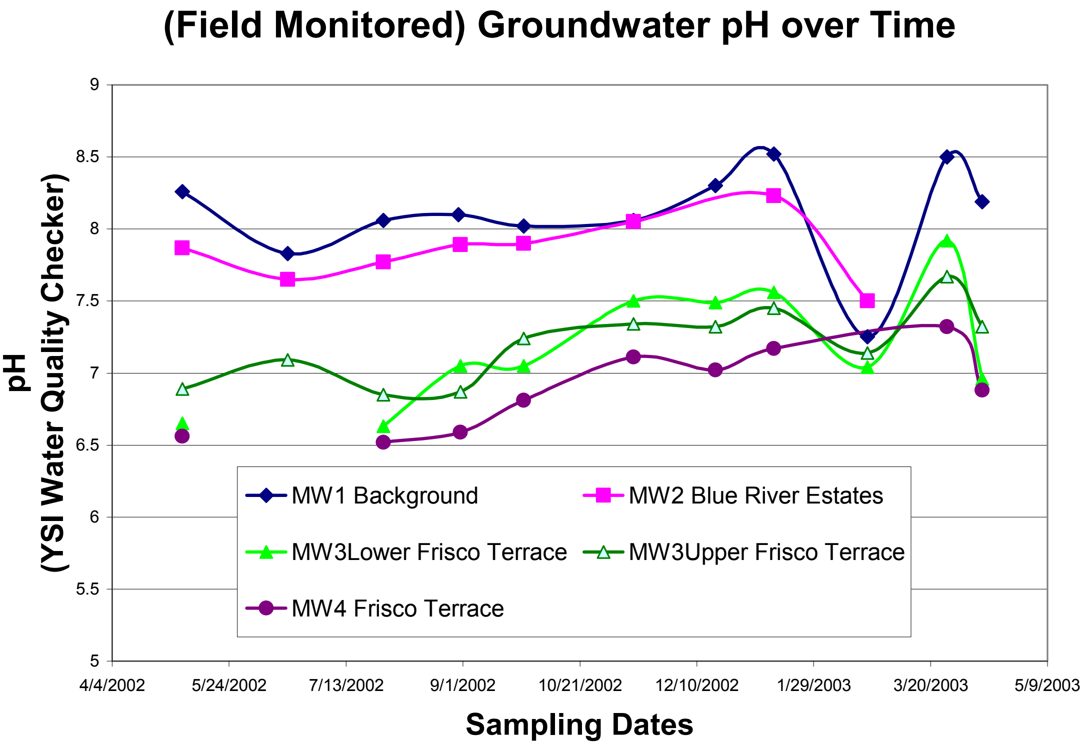


Figure 7. Field Monitored Ground Water pH

The total dissolved solids in ground water show increasing values as elevation decreases in the watershed (Figure 8). The highest TDS concentrations are found in MW3, in FT (closest to the Dillon Reservoir), while the lowest TDS values are found in the water of MW1 background and MW4. Factors contributing to this trend include: natural accumulation of weathering products during flow through the watershed and increased anthropogenic effects as the watershed is more populated at the discharge portion of the system. All wells seem to show a trend of increasing TDS in the late winter months of the sampling period.

Chloride concentrations (Figure 9) depict a similar trend as that of TDS, with increasing concentrations down the watershed flowpath. Chloride is a relatively un-reactive element as it passes through the subsurface, thus it is a good conservative tracer. The principle sources are atmospheric deposition and anthropogenic impacts. The temporal variations are very small in the background wells (MW1 and 2), but the wells near the bottom of the watershed (MW3 and 4) show a sharp short-term decrease during the winter (Dec-Jan). Anthropogenic sources of Cl include foods, salts, preservative, road salts in the winter for ice melt, etc. The elevated Cl concentrations seen in MW3 Upper in Frisco Terrace February and March 2002 may be a result of road salt impact as snow melt began early in the season in 2002, where as it is not seen in the corresponding months of 2003 due to the colder temperatures delaying snow melt during the 2003 season. Sampling during December -January (2002 and 2003) was difficult due to the thick layer of ice that had frozen over the ground surface in Frisco Terrace, not only making it difficult to access the monitoring wells (as the man hole covers were iced over with 3+ inches of solid ice, but also potentially acting as a barrier to surface water infiltration. This suggests that a major source of chloride in the groundwater is from the surface.

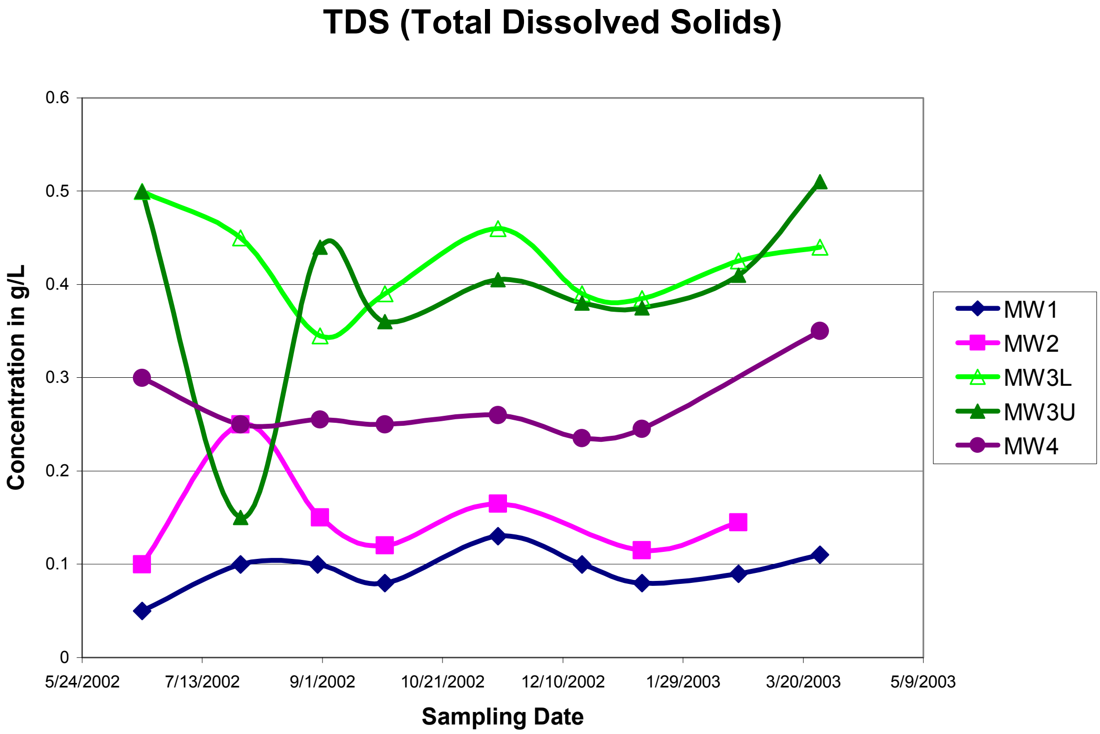


Figure 8. Total Dissolved Solids in Ground water over Time

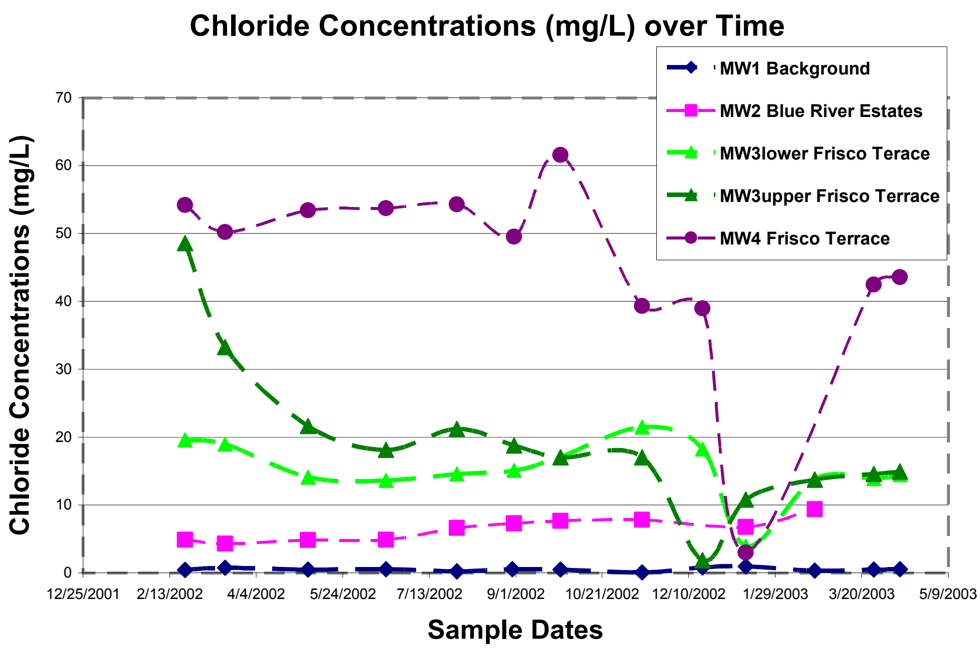


Figure 9. Chloride Concentrations mg/L in Ground water over Time

Nitrate concentrations (Figure 10) follow the trend of increasing concentrations down the watershed. Potential sources of nitrate can be either natural (found in the soils), atmospheric (rain or air), or anthropogenic (OWS). The likely explanation for the dramatic decrease in nitrate in the upper well of MW 3 and the smaller but corresponding dips in concentrations in MW2 and MW4 in the spring (June 2002) is dilution from spring snowmelt. Snowmelt for the winter of 2003 had not occurred as of the April.

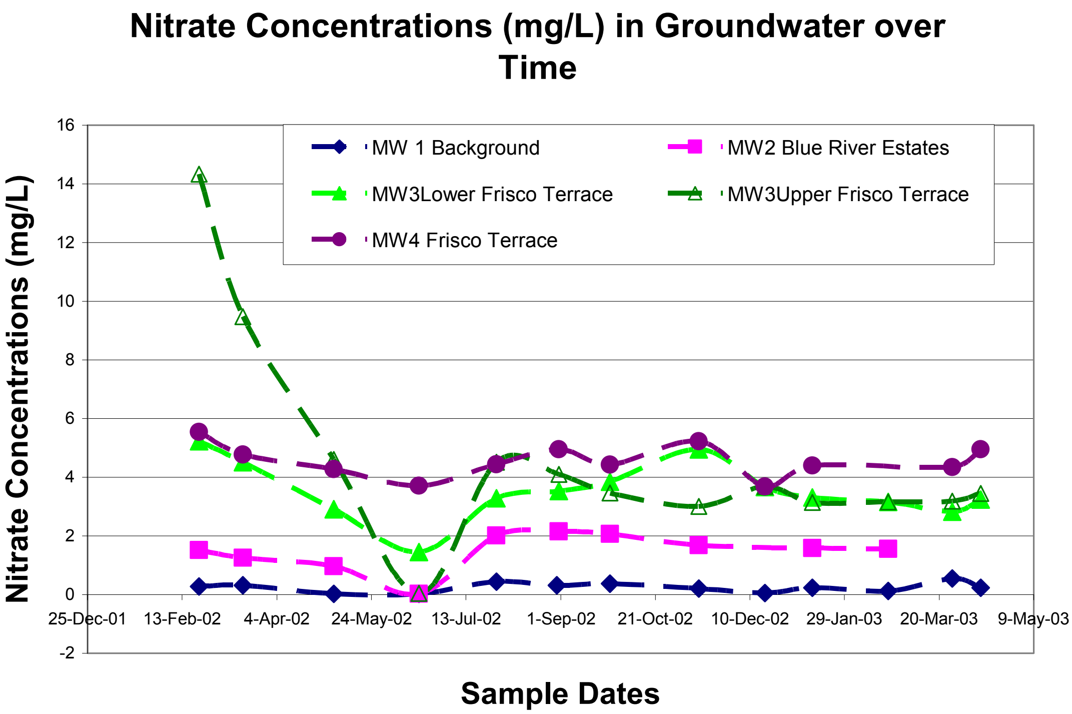


Figure 10. Nitrate Concentrations in Ground water

INTERPRETATION OF WATER-CHEMISTRY DATA

A total of 76 surface and ground water samples were taken along the Blue River drainage between September 2001 and August 2002. Analysis of the dataset showed the samples from the same location had little overall temporal variation except that noted above, but the samples did show significant spatial variations. Based on Na, K, Ca, Mg, HCO_3, SO_4, Cl, F and NO_3 data, the samples can be divided into six groups, arbitrarily numbered 1-6, each with distinct chemical signatures. The samples within a group are very similar to each other, but there is significant variation between groups. The chemical variations are primarily due to differences in sulfate, chloride and nitrate concentrations. Figure 11 is the Scholler plot of the mean concentrations for each group of samples.

Group 1 samples (n=7) are from the Pennsylvania Creek and MW-1. The water is primarily composed of Ca and HCO_3 with low total dissolved solids (TDS), typical of mountain drainages. This chemical signature is considered natural background for the Blue River drainage, without any significant anthropogenic influences. The similarity of the Pennsylvania Creek samples to

MW1 indicates that the MW1 is hydraulically connected to the surface water in the vicinity of the well.

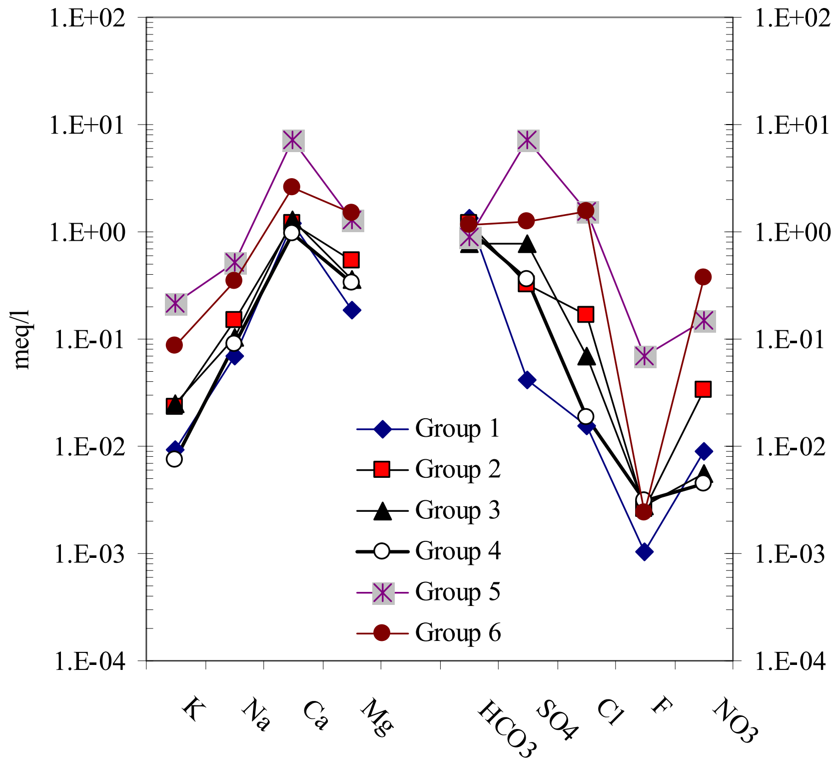


Figure 11. Scholler plot of the mean concentrations for water chemistry groups 1 through 6.

Group 2 samples (n=34) are from the length of Blue River and well MW2. These samples have higher TDS, are primarily composed of Ca and HCO_3, but have higher concentrations of SO_4, Cl, and NO_3 than background. Chloride and nitrate are anthropogenic indicators, consistent with the fact that the river flows through the towns of Blue River and Breckenridge. Sulfate is mainly derived from mineralized rocks and mine tailings. The similarity of the surface samples near the well and MW-2 water chemistry indicates that they are hydraulically connected.

Group 3 has only three samples, all stream samples taken from the confluence of French Gulch and Blue River. French Gulch, west of Breckenridge, drains a series of old mine workings. The group 3 samples are mostly composed of Ca and HCO_3, but have elevated SO_4 concentrations consistent with impact from ore minerals.

Group 4 samples (n=7) are stream samples from the Swan River drainage. The confluence between the Blue River and Swan River is downstream from French Gulch. The chemistry of the Swan River samples is very similar to those of group 1, with low TDS and solutes primarily composed of Ca and HCO_3 but slightly elevated SO_4 compared to the background samples of group 1.

Group 5 samples (n=21) are from the FT area and include surface water samples from North Tenmile Creek and ground water samples from well MW3. These samples have much higher TDS, significantly elevated SO_4 and higher Cl, F and NO_3 concentrations. The chemistry is interpreted as a combination of anthropogenic input and mine tailings influence. The elevated SO_4 is coupled with somewhat lower pH values and elevated trace metals such as Mn, Mo, Sr, Sn and Zn (Figure 13), a definite indicator of sulfide ore rock interaction. The similarity of the water chemistry between surface water and samples from well MW3 indicates that the surface water and groundwater are hydraulically connected.

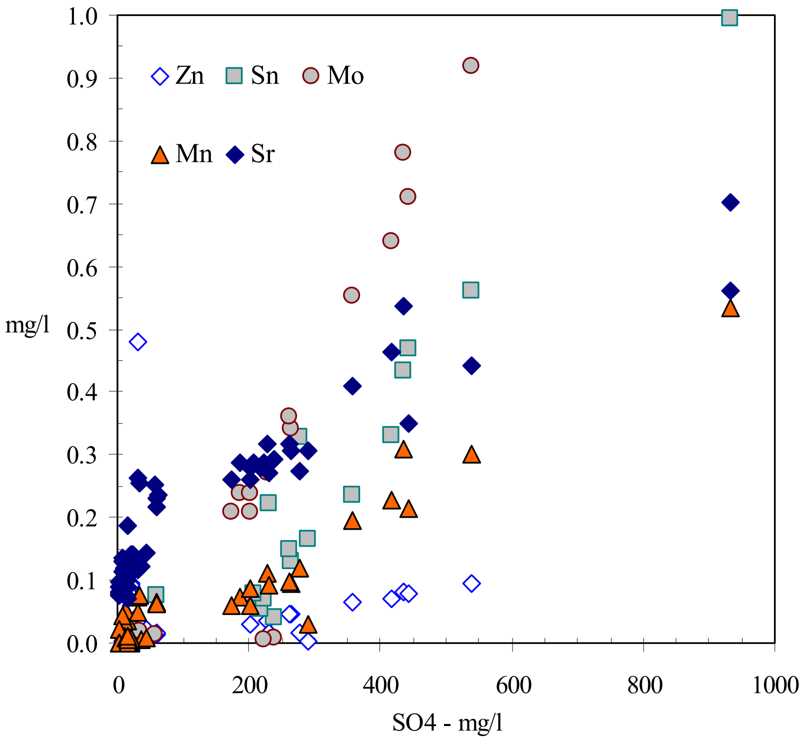


Figure 13. Plot of sulfate (SO_4) and trace metals in ground water and surface water samples. The data show increasing trace metal concentrations with increasing SO_4, which likely reflect weathering of sulfide ore rocks.

Group 6 consists of four samples from well MW4, which is located near MW3, but is more centrally located within the FT development. In spite of the spatial proximity to North Tenmile Creek and MW3, the well has a distinct chemistry with lower TDS and SO_4, but higher NO_3. This distinct signature indicates that the ground water in this well is not well connected to the local surface water system. However, it is possible that the ground water sampled by this well would be similar to the ground water at MW-3, but is mixed with soil-treated wastewater recharge with this high home density area.

SUMMARY

In conclusion, the water quality in the Blue River drainage varies in a predictable and systematic fashion. Surface water samples from the head of the Blue River, Pennsylvania Creek and Swan River show the chemical signatures of the natural water in mountain drainages. There is a clear chemical signature of elevated SO_4 associated with mine tailings. In addition, there is an anthropogenic signature that consists of elevated Cl and NO_3 content. Hydraulic head information indicates that all ground water sampled for this study is at a lower head potential than the nearby streams. However, chemical information suggests that the ground water and surface water are nearly similar. Wells MW1, MW2 and MW3 have chemical signatures that are identical to the local surface water systems. This implies that the ground water and surface water at those locations are hydraulically connected, and the surface water is likely recharging ground water. In contrast, well MW4 does not appear to be chemically well-connected to the local surface water system. However, this could occur because the ground water sampled by MW-4 is impacted by effluent from septic tanks. This is reasonable given the location of MW-4 within the FT development.

Based on this data we conclude that wastewater pollutants that migrate through the vadose zone to the ground water are not entering the nearby streams adjacent to the BRE and FT study areas because the hydraulic driving force is from stream to ground water. In the BRE area, it may be possible for pollutants to enter Blue River downstream from the study area into wetlands via a longer ground water flow path. At FT, ground water is likely discharging into Lake Dillon. The longer transport times provide more opportunity for soil-physical and biochemical attenuation of pollutants. For this study, monitoring stream concentrations just upstream and downstream of a development is not a useful method to assess whether septic tank pollution is entering the surface-water system.

The contaminant transport rates are potentially very fast at the BRE development. Thus, potential contamination of drinking water wells from septic-system wastewater could be a potential concern. However, the ground water in this area is low in nitrates and chlorides, indicating minimal wastewater impact to date. In FT, the ground water flow rates are slower, primarily due to lower hydraulic gradients, allowing more time for pollutants to be attenuated in the soil media before discharging to the Lake. This study clearly indicates that it is critically important to obtain site-specific soil-sorption and other contaminant transport parameters to obtain reliable estimates of pollutant transport times.

Finally, the study has demonstrated that ground water information from the CSEO well logs is useful in lieu of detailed monitoring-well data. These data are also likely to be very useful when designing a monitoring plan. However, integration of chemical and hydraulic data from monitoring wells and surface samples offer a more complete picture, particularly at the watershed scale.

ACKNOWLEDGEMENTS

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TECHNICAL HISTORY OF MANUFACTURED GAS IN COLORADO (1871-1940); BASIS FOR REMEDIAL ACTIVITIES

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Key Terms: manufactured gas, coal gas, water gas, coke, coal tar, PAH remediation, site characterization

ABSTRACT

Manufactured gas appeared in Colorado in 1871, the year that Denver was connected to transcontinental Pacific Railroad. As natural gas did not appear in Colorado for another 55 years and illumination by electricity was slow in coming, much of the sense of modernity and cultural achievement was quickly grasped by Coloradoans in the form of gas lighting.

Gas plants then quickly sprung up at the State's regional centers of commerce and mining, at Coal Creek (1879), Leadville (1883), Crested Butte (1884), Pueblo (1884), Colorado Springs (1889), Trinidad (ca. 1889) and Sterling (ca. 1891) and Georgetown (pre-1894).

The industry experienced opposition and consolidation (1883-1889), coal-tar chemical interests (1887), Pintsch railway oil gas (1891), and creosote wood treatment (1908). A brief supply of natural gas, from the Wellington field of northern Colorado, peaked and fizzled about 1928, but gas from Amarillo, Texas supplied the Front Range corridor by 1934. Carburetted water gas plants continued to operate elsewhere until the 1950s.

Carburetted water gas was introduced from 1913-1917, providing the first illuminating gas to several smaller cities. The author has discovered 65 gas works and associated coal-tar sites of Colorado. Denver leads with 15, followed by Pueblo with five.

INTRODUCTION

By definition, manufactured gas is made by pyrolytic (absence of oxygen) roasting of virtually any organic material so that its volatile content is driven off and captured, to serve as a gaseous fuel for artificial lighting, heating, or as a fuel (furnaces and smelters). The phenomenon of making gas from organic material was discovered at several locations, most notably in Britain and as well in France, what is now Czechoslovakia and in Germany, well within the 18th century. Commercial production of manufactured gas began at the opening of the 19th century, first at London (1812) and followed in the United States at Newport, Rhode Island (1810) and at Baltimore, Maryland (1816). The Chartered Gas Light & Coke Company of London is considered to be the first true commercial venture in manufactured gas. The American experience was crowned with a grand effort to have gas lights in place in all "modern" cities in time for celebration of the Centennial of 1876. Natural gas began to seriously compete with manufactured gas in the U.S. in 1880 and by 1966 had driven nearly all gas manufacturing out of business.

Today, former manufactured gas plants (FMGPs) of the U.S., when counted with their associated coal-tar distilling and derivative plant sites, may number more than 52,000 (Hatheway, 2003; USEPA, 1985). Many FMGPs and associated coal-tar sites represent environmental threats, both in terms of degradation of human health as well as of the environment.

GAS MANUFACTURING PROCESSES EMPLOYED IN COLORADO

Gas lighting swept across the West in the mid 19th century as an emblem of progress:

> "The sudden rise of new towns in the mining regions of Arizona will soon necessitate the construction of gas works. In California, gas-works were built, in increasing numbers, soon after the mining business became a settlement thing, and also in Nevada. As a large influx of emigration is now settling in Arizona, new towns and villages must spring up with great rapidity, and the people will have the best artificial light. The same remark will also apply to Colorado, which is rapidly being settled."
>
> (*American Gas-Light Journal*, 15 Apr, 1864, p. 311)

From earliest to generally latest, Table 1 presents the gas-making processes that are known or suspected to have been employed in historic Colorado undertakings. An example of a gas holder house, one of the components in production and delivery of the gas, is shown in Figure 1.

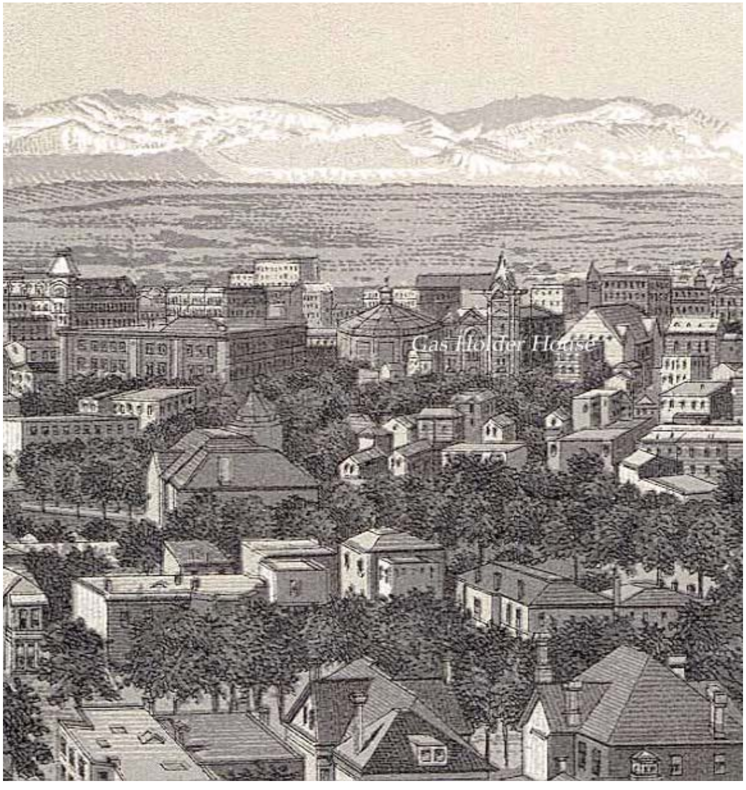

Figure 1. Lithographic view of a segment of the Denver Central Business District, toward the west-northwest across the South Platte River and the rail yards showing a gas holder as enclosed within a gas holder house (see label, above) with its distinctive false windows and broad, conical roof in the uppermost central portion of the view, 1890 (From a Union Pacific Railroad brochure of that year).

FORMER MANUFACTURED GAS PLANTS OF COLORADO

The author has compiled a chronology of manufactured gas in Colorado (Appendix A) and a list (Appendix B) of former manufactured gas plants of Colorado, as discovered through review of technical literature, Sanborn Fire Insurance Maps, and records held by the State of Colorado. Additional information was gleaned from *Brown's Directory of North American Gas Companies* (yearly from 1889), journals and gas association proceedings, and newspaper accounts. The USEPA published a limited review of the entire years of coverage of Brown's Directory, in 1985.

The list contains 72 sites, of which all but a few are FMGPs, the remainder being related "coal-tar" sites with virtually the same environmental threats. A 1997 technical paper (Hatheway, 1997) contains a listing of the many types of associated sites at which one may expect to encounter polycyclical aromatic hydrocarbons and other FMGP-similar toxic wastes.

SUMMARY AND CONCLUSION

Colorado was the scene of at least 55 separate manufactured gas plants, producer gas plants, Pintsch railway gas plants and coke ovens plants. Additionally, another 13 coal-tar sites of coke-oven works, tar by-product plants such as creosote and chemical tar distilleries, tar-paper factories, coal-tar dye works, wood-treatment plants, oil-shale retorts, and light-oil solvent plants have been identified by the author. Producer gas plants are expected, but have not been positively identified at the larger mines and smelters and at industrial plants at Denver, Pueblo, Colorado Springs and Grand Junction.

Colorado is faced with a unique threat in the form of its abandoned and now forgotten former manufactured gas plants. There is no master listing of such sites either nationwide or by individual States. Paramount to the issue are conclusions that many of the tar wastes are known carcinogens (yet most have not been studied in detail) and coal tars are non degradable and are persistent in the environment.

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Table 1. Tar-Yielding Processes Known or Suspected to Have Been Utilized in Colorado

Process	Plant Physical Characteristics	Potential Toxic Residuals
Charcoal	Beehive ovens of brick or stone masonry used to roast rood in the absence of oxygen.	Produced much smoke made up of organic volatiles released as gases, but also released wood tars containing PAHs.
Wood Gas or Gas Made from Organic Plant Matter	Required the same retorts used in coal-gas generation. Often the choice at small-town or remote locations prior to 1890.	Produced mainly from resinous wood (fatwood). Left virtually the same assemblage of toxic residuals as produced from coal as a feedstock.
Coal Gas	The fundamental historic gas manufacturing process. Gas plant consists of benches of multiply-heated retorts for roasting of coal to release gas. Typical small town gas works made 3,000,000 – 10,000,000 cf per year.	Coal volatiles driven off as gas; tar, ammonia, sulphur, cyanide and heavy metals (mainly arsenic) removed through various "clarification" and "purification" processes. Tar had value under some circumstances or could have been burned at the plant as fuel; under other circumstances tar was dumped around the plant. Spent purifier waste nominally was dumped.
Illuminating Oil Gas	Briefly utilized at Colorado Springs as the first commercial demonstration of the Leon P. Lowe process developed at Lynn, Mass.	All matter of natural mineral oils and refined oil fractions have been suitable to produce illuminating gas.
Carburetted Water Gas	Invented 1873 by Civil War balloonist and father of U.S. Air Force, T.S.C. Lowe. Process required three vertical steel cylinders much different than coal-gas retorts. Steam was flashed to H2 and CO and then enriched with atomized light oil for use in gas lights.	Residuals similar to those of coal gas, but without ammonia and of lesser amounts of tar and cyanogens created per thousand cubic feet of gas produced.
Producer Gas AKA "Blue Gas" (Fuel-Type Water Gas)	Fuel gas produced by roasting coke and injecting steam. Made up of carbon monoxide and hydrogen and releasing enough Btu to serve to heat for industrial purposes; properly known as "blue gas" and insufficient illuminating quality for lighting purposes.	Common to heat-consumptive industries, especially ore smelting plants, many of which were built after 1890 at major mining districts, such as Leadville, and at smelter centers such as Pueblo. Toxic residuals similar to those of coal-gas plants.
Oil-Enriched Water Gas	Water gas enriched with a variety of injection processes devised to circumvent Lowe's patents. After ca. 1892 enough of Lowe patents had expired and numerous manufacturers offered "knock-off" and improved Lowe-type water-gas sets.	Residuals identical to those of carburetted water gas plants.
Pintsch Gas	1883: Introduction to U.S.; tightly patented German process for illumination of railway cars. Pintsch plants typically located at all railroad division yards. Obsolete by 1940.	Residuals were basically similar to those of oil gas and carburetted water gas.
Lowe Double Standard Carburetted Water Gas Generating Sets	Developed by the United Gas Improvement Company of Philadelphia, from patents of T.S.C. Lowe, as purchased in 1884. Increased daily yield from 50,000 cf to as much as 1,000,000 cf. Utilized at Denver.	Residuals identical to those of standard carburetted water gas production.
Bottled Blau Gas	Manufactured enriched water gas or solvent-vapor Illuminating gas Compressed to liquid state into small, portable cylinders for rural or suburban use.	Not yet identified in Colorado, but likely to have been present at least along the Front Range, on the basis of general population locations. Residuals similar to those of carburetted water gas.

APPENDIX A: Chronology of Gas Manufacturing in Colorado

1871-1880: Coal Gas Dominates

Considering the widespread availability of coal in the United States at the time that Denver had just been connected with the Transcontinental Railway at Cheyenne, and that Lowe's carburetted water gas apparatus was not yet fully patented, it is likely that Denver's first gas works made use of coal gas generation.

1889: Oil Gas Appears, Water Gas Rises; Coal Gas Dominates.

Leon P. Lowe's oil-gas conversion of the Colorado Springs gas works was Colorado's outstanding contribution to the national development of manufactured gas in North America. It is likely that Lowe's operation was able to use crude oil as the feedstock, though this has not yet been proven, and that his oil arrived via railroad. Lowe went off to California within the year but remained financially involved with the plant for some years. Without his presence as a promoter of oil gas, the process does not seem to have spread further in Colorado or surrounding States. Lowe, however, became the prime mover in the creation of the Pacific Coast Oil Gas process that swept California by 1900 and eventually led to nearly 300 gas works (Hatheway, 1999).

1890-1930: Producer Gas Plants at Mines and Colorado Factories

During this period, gas plants were widely installed in plants, shops or agencies in which fuel was used to create some form of energy to drive equipment. In Colorado, for example, the small town of Rocky Ford had its own municipal producer gas plant for the purpose of driving the pumps that supplied drinking water. In 1914, Buena Vista had a plant making producer gas engines, likely for various agricultural and mining uses. These plants generated residuals similar to those of carburetted water gas and have frequently been found to be contaminated by hazardous waste.

1927: Natural Gas Appears in Colorado

Limited natural gas was found in Colorado, for the first time, at the Wellington field in the north fringe of the State in 1927. This gas briefly was used to serve Fort Collins, as well as Cheyenne, Wyoming.

1934: Natural Gas Arrives at Denver

Natural gas delivery was delayed by the Great Depression late in 1929. The readiness in 1927 had been largely conditioned by advances in metallurgical engineering allowing for the butt-welding of seamless pipe, capable of handling high-pressure transmission. The year of disappearance of manufactured gas from Colorado has been picked by the author as 1940, based on fragmentary evidence related to the existence of manufactured gas at the widely-dispersed non-Front Range corridor population centers of the State, notably Grand Junction and Sterling.

APPENDIX B: List of Known Manufactured Gas Plants in Colorado

The table below constitutes the author's research in identifying and recording separate gas manufacturing and related "coal-tar" sites of Colorado. The list represents a dedicated attempt to locate and record the founding and existence of all manner of former industrial sites related to the technology of manufactured gas and current presence of its non-degradable toxic wastes. No claims are made as to the completeness of the list.

Location	Origin	Owner	Features
Aguilar, no. 1	1902	Trinidad Electric Transmission Railway & Gas Company Also Northern Coal & Coke Co. interested at the time	Undetermined authenticity
Aguilar, no. 2	1902	National Coal & Coke Co. proposes to build a gas works	Unknown
Aspen-Glenwood Springs	Unknown	Beehive coke ovens Unknown owner	
Boulder, no. 1	1902 1914	Promoter Peter English; Later Federal Gas Co. (Natural gas) Federal Gas Co. purchased by General Service Co.	1915: Federal Gas Co. installs carburetted water gas set for peak service on failure of gas wells 1930 Natural gas arrives 1956: Demolition of plant
Boulder, no. 2	1903	John C. Conway, Buffalo, NY	To serve natural gas from National Consolidated Oil Co.
Bowie	1921	Juniata Coal & Coke Co.	Coke ovens for mining & smelting production
Buena Vista	1914	Establishment of the Kuenzel Process Gas Generator & Smelting Co., of Mexico	Manufacture of producer gas generators for use in brick kilns and glass works
Burlington	1916	Burlington Gas & Electric Co.	Acetylene gas plant, for town lighting on subscription 1919
Canon City	1903	Establishment of Canon City Gas Co	Carburetted water gas; operated through most of 1940s
Cardiff	1919	Coke ovens in place.	Likely beehives, without capture of tars or PAHs
Castle Rock	1916	Castle Rock Light, Heat & Power Company	
Center	1914-1916	Center Gas & Light Co.	No details available
Claremont	1903	Citizens' Light & Water Co.	Gas plant contemplated
Coal Creek	1879 1891	Establishment of the Colorado Coal & Iron Company 30 new coke ovens 1891: Produced 121,657 tons coke	Beehive coke ovens, likely without capture of tars & PAHs
Colorado City	1917	United Gas & Electric Engineering Corp.	No details available

Location	Origin	Owner	Features
Colorado Springs, no. 1	pre-1889	Colorado Springs Gas & Electric Company	Coal gas plant
Colorado Springs, no .2	1889 1912 1925	Colorado Springs Lowe Gas & Electric Company 1894: Company sold to local interests Later: Colorado Springs Light, Heat & Power Co. Colorado Springs Light, Heat & Power Company Colorado Springs Gas Company	1894: Plant augmented with 5-ft CWG set. May have been owned by Leavenworth, KS Gas Co.
Colorado Springs, no. 3	1925	Apparently a municipal gas plant	4,954 customers
Crested Butte	1884 1891	Colorado Coal & Iron Co. 30 new coke ovens	Likely date of establishment of beehive coke ovens
De Beque	1916	Rifle-DeBeque Oil & Gas Co., of DeBeque	Presumed manufactured gas works; Reported to PUC
Denver, no. 1 Off NE cor 18th & Wayzee	1871	Denver Gas Co.; Established by "Col." James Archer, St. Louis, MO 1884: Company modernizes and expands its coal gas plant	Year after arrival of rail service at Denver
Denver, no. 2 Undetermined Location	1883	Opposition gas company established post-1883: Denver Consolidated Gas Co. takes control	Denver Consolidated Gas Co. survives 1889: Combined company makes 300,000,000 cf coal gas and CWG
Denver, no. 3 Undetermined Location	1887 1890 ca. 1920	Western Chemical Manufacturing Co. General Chemical Co. Allied Chemical Co.	One coal-tar dye manufacturing plant, under various owners
Denver, no. 4 N. cor. Blake & 20th Sts.	1890	Gas Holder	Pit-type, with subsurface tank
Denver, no. 5 Undetermined Location	1891	Pintsch railway oil gas plant installed Total output is 300,000,000 cf/yr	Both coal gas and Lowe carburetted water gas
Denver, no. 6 6th & Lawrence Sts.	pre-1901	Gas Works	Undergoing improvement
Denver, no. 7 40517th St., AKA Wewatta St., between 7th & 8th Streets	pre-1903	L.A. Watkins Tar Paper Factory	Using gas-works residual tar
Denver, no. 8 Off Mill Creek	1903	New coal-gas plant Denver Gas & Electric Company	Sanborn Map shows new 1,000,000 cf gas holder
Denver, no. 9	1903	Denver Ammonia Works	In place; Used gas liquor
Denver, no. 10 Vicinity of Blake Street between 18th & 19th Streets	pre-1905	Gas holder	

Location	Origin	Owner	Features
Denver, no. 11 881 S. Emerson St.	1905	Denver Gas & Electric Co.	Unidentified facility
Denver, no. 12	post-1905	New Pintsch railway oil gas plant	Proximous to Union Depot
Denver, no. 13	1908 in place	Denver City Tramway Co.	Open-dip wood treatment
Denver, no. 14	1918	Unidentified oil shale retorting company	Employed dental furnace as pilot plant, for western Colorado oil shale
Denver, no. 15	1918	U.S. Government Toluol Plant Co-located with Denver Gas Co.	WWI Emergency Measure
Denver, no. 16	1919	Crown Tar Works	Subsidiary of Denver Gas & Electric Co.
Denver, no. 17 Undetermined Location	1924	New, high-technology gas manufacturing plant installed	
Denver, no. 18	Unknown	Broderick Wood Products Co.	USEPA SUPERFUND NPL creosote wood treatment site
Durango, no. 1	1899	Durango Electric Light Co Durango Gas & Coke Co. 1905: Durango Gas & Electric Co is incorporated	Receive franchises for gas plants 1905: Sanborn Map shows gas works for first time
Durango, no. 2	1902	Incorporation of San Juan Coke & Gas Co. to erect a ten-oven by-product coke battery	
El Moro, Las Animas County	1888	1879: Colorado Coal & Iron Co. established Capitalized at $10,000,000 1888: Likely date of establishment of beehive coke ovens	Coke ovens 1888 total production from this works and at Crested Butte = 137,482 tons coke
Fort Collins	ca. 1902	Established by Poudre Valley Gas Co. 1914: Taken over by General Service Co. 1925: Brief service by natural gas from Wellington field of n. Co.	Oil-gas plant 1946: Was a coal gas plant of Public Service Co. Col. 1948: Plant demolished save for gas holder
Georgetown	pre-1894	Established by Georgetown Electric Light & Power Co.	Planned shut-down, yet gas plant still listed in 1899 @ 1,500,000 cf/year
Goldfield, El Paso County	1894	Goldfield Water, Gas & Electric Light & Power Company	Incorporated @ $100,000
Grand Junction	1906	1906: Established as Grand Junction Electric, Gas & Manufacturing Company 1926: Grand Junction Electric, Gas & Manufacturing Co. 1926: Company taken over by Public Service Co. of Colorado	Located at 825 Flint Ave/ as Two parcels One active In VCP One requested for “No Action Document” Known to have operated at least through 1926
Greeley, no. 1	ca. 1904	Greeley Gas & Electric Co. 1909: Plant had been demolished	Apparently had two Pintsch oil gas generators

Location	Origin	Owner	Features
Greeley, no. 2	ca. 1916	Greeley Gas & Fuel Co. pre-1943: Plant demolished	Four coal-gas benches and two gas holders
Gunnison 0.5 mi NW, La Veta Hotel	pre-1899	Gunnison Gas & Water Company	1899; *Brown's Directory*; gas works "closed"
Ivywild	pre-1917	United Gas & Electric Engineering Co.	One of the largest of the national holding companies
Kersey	ca. 1914	Home Gas & Electric Co..	Large national holding company
La Junta, no.1	ca. 1914	Otero Gas Company; Masseurs. Joy and Culver, Denver	1928: Was using Beale Oil Gas process 1930: Likely defunct
La Junta, no. 2	1922	E.F. Chambers Gas Company	Listed by CO PUC
Las Animas	pre-1903	Las Animas Light, Power & Mfg. Co.	Company headquartered at Trinidad, Colorado
Leadville	pre-1883	Leadville Gas Work Leadville Illuminating Gas Co.	1899: 16,000,000 cf/year coal gas pre-1937: Inactivated
Mancos	1900	Citizen vote on motion to allow franchise for private gas company to operate in the town	Outcome not determined
Monte Vista	pre-1890	Monte Vista Electric & Gas Light Company	Plant closed pre-1920
Poudre Valley	pre-1914	Poudre Valley Gas Co. established, builds, operates gas works	1926: Gas company and gas works acquired by Colorado Public Service Co.
Primero	pre-1922	Colorado Fuel & Iron Company	Was operating dip-type wood preservation plant
Pueblo, no. 1	1884	Incorporation of Central Pueblo Gas Light Co. 1902: Control to Pueblo Gas & Fuel Co. 1928: Owned by Federal Light & Traction Co.	1910: Using CWG process 1920: 140,000,000 cf/year 1928: Natural gas arrives in Pueblo from Amarillo 1930: Gas plant likely was inactive 2003: Logged into VCP 615 West Street
Pueblo, no. 2	ca. 1901	Colorado Fuel & Iron Co. Producer gas plant	Duff gas producers
Pueblo, no. 3	pre-1908	East Pueblo Fuel Co. granted franchise	To serve the east side of the City
Pueblo, no. 4	1916	Inspiration Copper Company Smelter	Taking tar produced by Pueblo Gas & Fuel Co.
Pueblo, no. 5	ca. 1917	Colorado Fuel & Iron Co. Minnequa Plant	Two batteries of 60 Koppers by-product coke ovens at the existing steel plant 1946: 262 coke ovens
Pueblo, no. 6	1918	Toluol Refining Plant	Denver Gas & Electric Co.
Pueblo no. 7	Unknown	Poleyard I (11th St.)	2003: In VCP\
Pueblo no. 8	Unknown	Poleyard II (11th St.)	2003: No Action Approved
Redstone	ca. 1919	Unknown operator; Likely Colorado Fuel & Iron Co.	Coke ovens

Location	Origin	Owner	Features
Rifle Area	1970s	Various oil-shale gasification pilot plants	Predicted to have generated and released PAHs
Rocky Ford, no. 1	ca. 1914	Municipal gas works	No other details
Rocky Ford, no. 2	ca. 1916	Municipal water works	Pumps powered by on-site producer gas
Saguache	ca. 1916	Saguache Gas & Electric Co.	No other details
Sterling	ca. 1891	Establishment of unknown gas company 1909: Bought by Colorado Power Co.	1924: Bought by Public Service Co. of Colorado 1998: Generator house still standing
Trinidad	1889	Trinidad Gas Co. was in place 1916: Owned by Trinidad Electric Transmission Railway & Gas Co., Springfield, MO	1928: Was a holding of Federal Light & Traction Co 1950: Plant was inactive
Unknown	1970s	Occidental Petroleum Company	Coal gasification plant

THE CHALLENGES OF MINING IN COLORADO

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ABSTRACT

From the 1859 gold rush to today's technologically driven large-scale mines, Colorado's mining industry has faced and overcome numerous geological challenges. The record size blasts in the Climax mine and the billions spent on oil shale development are two unique aspects of Colorado's mining industry. Substantial obstacles have been overcome to extract Colorado's incredible wealth in the form of metals, coal, and industrial minerals. The diligent application of science and engineering has been a requirement for successful mining operations in the Mountain State. Colorado's mining industry has a record of achievement including developing a transportation system, developing extraction techniques and markets for new commodities, dealing with ground water inflows, cold weather operations, and acid mine drainage. Today, reclamation of abandoned mine lands is restoring Colorado's landscape while furthering the State's mining and geological engineering technology capabilities.

INTRODUCTION

Mining activity led to the establishment of cities and towns in Colorado and continues to leave its mark upon the landscape. Over 46 new mineral species have been discovered in Colorado and it was here that such important commodities such as uranium (1898) and molybdenum (1916) were first produced on a commercial scale (Eckel, 1961). In addition to the many metals such as gold, silver, lead, tungsten, and zinc produced in Colorado, the State has also been a source of rare metals like vanadium, industrial minerals including clays, gypsum, light weight aggregates, limestone, sodium compounds, and marble, and gems such as aquamarine, turquoise, and recently diamonds. The diversity in mineral wealth is matched by the diversity of scientific and engineering expertise that has been developed to extract it. Colorado has contributed much to the advancement of mining and mineral processing and continues to export this expertise to the world.

One example of this expertise is that of the development of the geologic compass. In the 1880's David W. Brunton devised systematic geological mapping techniques at Aspen, Colorado. The Aspen mine owners sent him to Butte, Montana to offer his services to a small mine owner who was involved in litigation over the apex of a faulted silver vein. The Butte case was very similar to litigation involving the Aspen silver mines. A loss in Butte would be a bad precedent. Brunton's mapping was so convincing that the case was quickly settled out of court. Brunton was hired as a mining consultant to the Anaconda Company. Reno Sales, the father of mining geology, learned how to perform geologic mapping from his boss David Brunton. Returning to

Colorado, Brunton perfected his geologic compass and obtained a patent on the device. In 1904 he hired the Ainsworth Instrument Company of Denver to manufacture and sell the device. Although the compass is now manufactured in Wyoming, to this day every geologist is trained to use a Brunton compass.

HISTORY OF MINING IN COLORADO

The Gold Rush

In 1858 a group of men from Georgia decided to return to western Kansas Territory where they had previously found small amounts of gold during a winter stop during the California gold rush. Led by William Green Russell, the group set out across the Great Plains in July 1858 under the escort of Captain Lyon and 20 soldiers (Rickard, 1932). The prospectors found their old camp at the junction of Cherry Creek and the Platte River. The men split up, one party went north finding a creek full of boulders. Gold was found on a branch of Boulder Creek near Gold Hill. Another party went across the mountains and found rich gravel at a place they named Russell Gulch in honor of their leader. The gold was given to six of the men who were sent east to obtain better tools and more provisions. Russell also sent a letter to some of his relatives in Georgia, telling them to come and share in the rich gold find. Rumors of the gold find spread over Kansas. A number of men came to Cherry Creek in the fall of 1858, establishing a settlement they called Auraria. George Jackson went into the mountains in the winter finding hot soda springs and nearby he discovered rich placers at the mouth of Chicago Creek. Returning to Auraria (Denver) in April, 1859 Jackson offered his gold for tools and provisions. This touched off an intense rush into the mountains and word of the find made its way East. Mr. D. C. Oakes, who made a good discovery, went back East for supplies and while there published a guide to the gold fields. The second great American gold rush was on.

On May 6, 1859 John Hamilton Gregory made the best find of all. The Gregory lode was a vein that crossed the gulch and contained many pieces of weathered rock covered with gold. He took out pounds of gold. He was kind to newcomers showing them how to prospect. In his tent he kept a trunk under his cot filled with large specimens of gold that he enjoyed showing people. Gregory sold two claims in June of 1859 for $21,000. By July of 1859 there were 100 sluices and 1,000 men working the area around Gregory Gulch and the towns of Central City and Black Hawk were soon established.

The Pike’s Peak gold rush was fast paced, about 15,000 came to the Front Range area by the fall of 1860 and another 10,000 were at Oro City (Leadville). The plains were filled with people and animals including oxen, horses, donkeys, and cows packing supplies and pulling wagons, buggies, carts, and wheel barrows, while others walked the last six hundred miles of the journey with their outfits on their backs (Thayer, 1887). Personalized license plates on automobiles are nothing new, the gold rushers’ wagons were painted with slogans such as “Lightening Express”, “Pikes Peak or Bust” and hundreds of other slogans (Figure 1). By summer it was a constant stream of people. Some were disappointed that they could not just pick up gold nuggets from the streams.

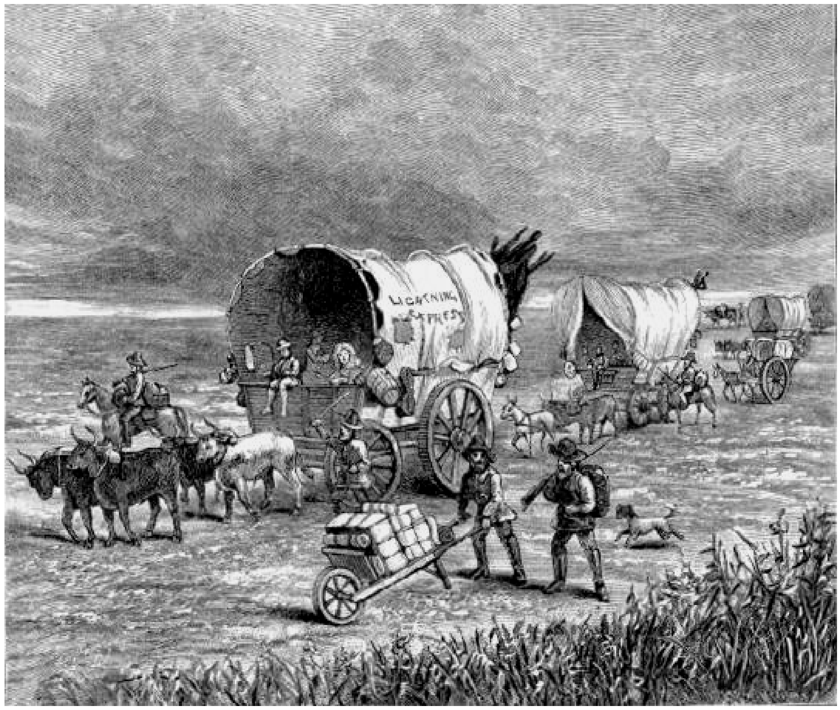

Figure 1. Crossing the Great Plains in 1860. [Thayer, (1887)].

Others mistakenly climbed Pike's Peak and beyond, digging holes in the hillsides looking for gold in barren rock. Only a handful of prospectors like Jackson and Gregory were so lucky as to scoop gold out of the ground with minimal effort. Those unwilling to perform the hard labor that most placer mining required decided to go back. For two years the rushers and the "go backs" passed each other in their journey across the plains. Several hundred died in the journey from starvation, thirst, disease, and Indian attacks. People learned to travel in groups and carry large amounts of food and water.

Colorado Territory

By the summer of 1860 prospectors had spread all over Colorado and tested nearly every river, creek, and gulch for placer gold except for the San Juan Mountains where the government had not yet made a treaty with the hostile Ute Indians. Half of Colorado's mining districts were discovered. Auraria (Denver), Central City, Golden, and Idaho Springs were established centers of population.

One big find was near the headwaters of the Arkansas River. On April 6, 1860 John O'Farrel and his party stopped at noon. He went to the creek to get some water for his coffee. Upon breaking through the snow and ice he found gold lying on the sand bar. The men began working the area. A few days later Abe Lee exclaimed "boys I got all of California here in my pan!" Horace Tabor and Samuel Kellogg came by on April 26th and in two months time took out $75,000 in gold from their claims. Oro City was the name of the new town at California Gulch where $1 million in placer gold was recovered that first summer. This was the beginning of the Tabor fortune at what would later become Leadville.

On February 26, 1861 Congress created the Territory of Colorado and Major William Gilpin was appointed Governor. The rich placer yields declined and hardrock mining took off as heavy equipment was brought in and stamp mills were erected.

All total, Colorado's gold production amounts to about 46 million ounces, 22 million of this coming from Cripple Creek. W. S. Stratton's Independence mine was the most notable property at Cripple Creek. Other big gold producing districts include Central City with 4.2 million ounces, Telluride with 3 million ounces, Leadville with over 3 million ounces, Breckenridge with 1 million ounces, and Summitville with 0.5 million ounces.

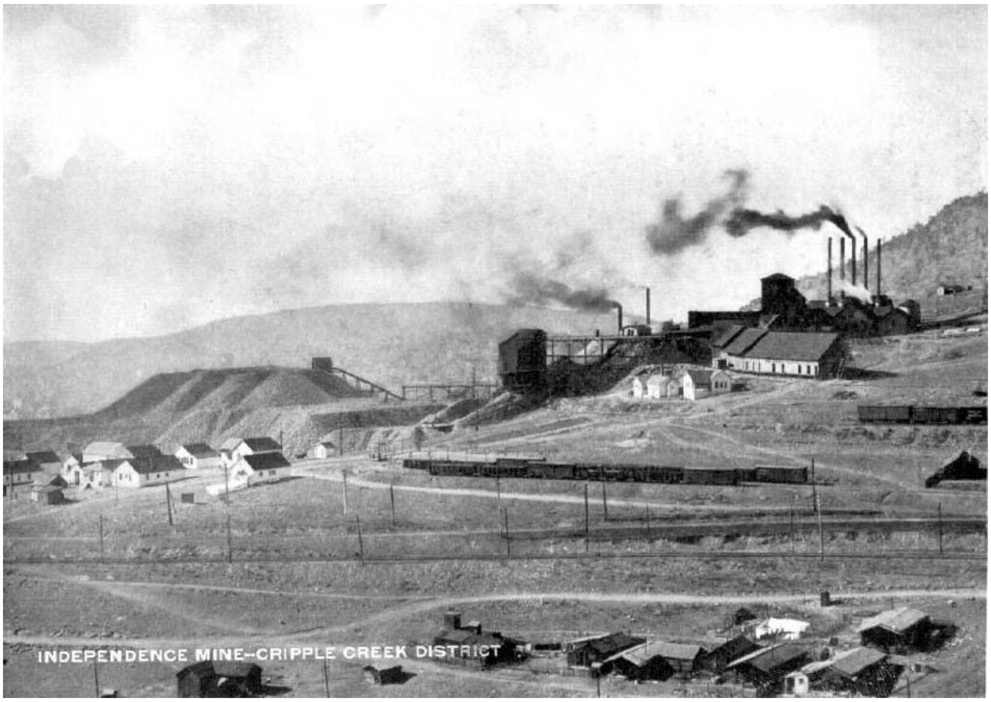

Figure 2. W. S. Stratton made over $2 million in profit from his Independence mine.

The Silver State

It was the silver mining era that industrialized Colorado. John Coley first discovered silver in at Saints John near Montezuma in 1863. In 1865 the Saints John mine was developed by the Boston Silver Mining Company at Coley's discovery.

James Huff staked the Belmont lode on September 14, 1864. A few pounds of samples from the discovery were taken to Central City where it assayed from $200 to $500 per ton in silver. The Pine Silver Mining Company was promptly formed and the Argentine district was established.

Hardrock mining required heavy equipment, rails and mine cars, hoists, mills, smelters, and coal for fuel. In 1868 Nathaniel Hill erected a smelter at Black Hawk, Colorado. There were few good roads and transportation was the first major challenge the hardrock mining industry faced. Enterprising pioneers met this challenge. Men constructed toll roads such as Ute Pass (Figure 3) installed for $15,000 as a route to Leadville in the 1870's. There were three toll roads to Leadville and one to Black Hawk.

The railroads quickly followed. In 1870, Denver was connected to the Union Pacific Railroad by a line constructed from Cheyenne, Wyoming. The railroads wanted access to Colorado's emerging wealth. A competing line was completed the same year by the Kansas Pacific. The Colorado Central was a narrow gauge railroad that connected Denver with the silver and gold mines of Black Hawk in 1872. The line was extended to Central City in 1878. By 1880 the Denver and Rio Grande reached Leadville. Mountain railroads required engineering of rock cuts, bridges across rivers and streams, and tunnels through high peaks. A decade later, Colorado had a vast network of mountain railroads that allowed travel to remote mining towns.

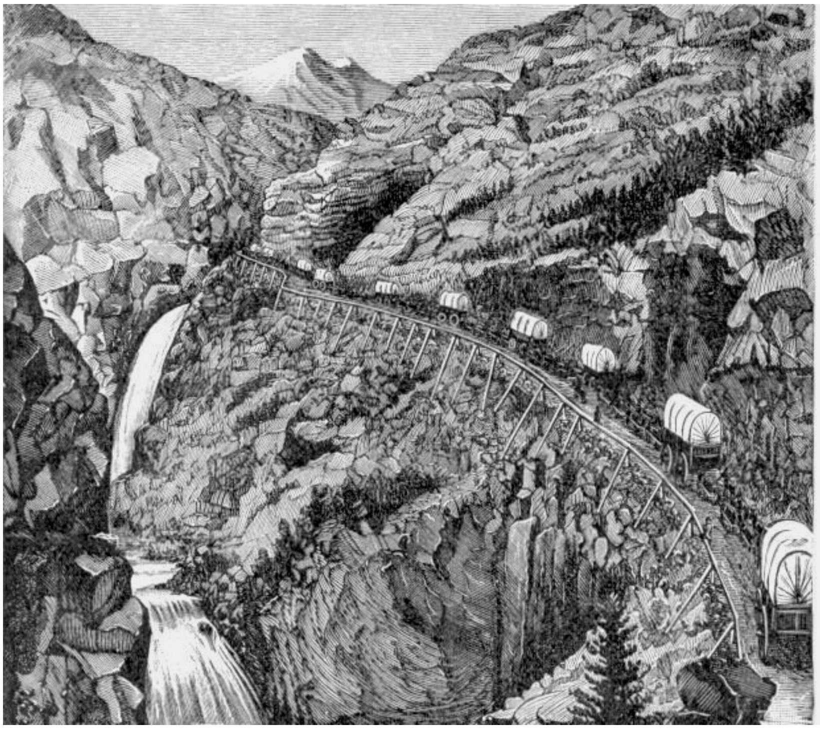

Figure 3. The Ute Pass toll road was constructed for $15,000 [Thayer, (1887)].

Figure 4. The Devil's Gate Viaduct above Georgetown was built 180 degrees backwards and did not fit the abutments properly. It was rebuilt in six weeks being completed January 23, 1884.

The big silver strikes were at Leadville in 1874, Aspen in 1879, and Creede in 1889, but rich silver mines were operated across the state. By 1880 Colorado had thousands of silver mines feeding hundreds of mills and more than a dozen smelters.

Not all Colorado ventures were successful. Mark "Brick" Pomeroy championed the Atlantic-Pacific tunnel, a proposed 25,200 ft. (7,700 m) boring through Gray's Peak (Eberhart, 1974). It was to be both a mining and a transportation venture. The tunnel would provide a short route to Leadville and in the process of its construction would cut through approximately 200 fissure veins at depth. In 1890 shares were sold for $2.50 each and construction was initiated. By 1892 4,000 ft. (1,200 m) from the east and 1,400 ft. (427 m) from the west side were excavated and a few promising veins of ore were uncovered. The silver panic of 1893 stopped progress and the company went bankrupt. It resumed in 1896 only briefly, and again in 1905 a few more thousand feet was advanced into the mountain before it finally fell silent.

Industrialization

Colorado has contributed greatly to the advancement of the mining industry. Hundreds of innovations have come from the state in the form of advancements in mining equipment and metallurgical processes. The following innovations have been applied worldwide subsequent to their development in Colorado. The world's first commercial electric powerplant used an alternating current power system designed by Nicholas Tesla. It was installed near Ophir, in southwest Colorado, in 1891 to power the Gold King mine. The Ames powerplant at Ophir is still in operation today as part of the State's public utility system. In 1893 the first electrically powered mine hoist was installed at Aspen, Colorado. Today, nearly all mine hoists are electric powered.

In 1940 Harold F. Silver developed the continuous miner, a carbide-tipped machine that can dig and load coal without blasting (Mining World, 1959). It was installed at the Consolidated Coal and Coke Company's Baum mine at Frederick, just north of Denver. The rights were later sold and today it is used all over the world under the name of the Joy Continuous Miner.

The use of chain link fabric in conjunction with rock bolts to support fractured rock is a Colorado idea developed after World War II by engineers from the Colorado Fuel and Iron Company. This technique is used all over the world in mines and is also used to support many highway rock cuts.

Molybdenum

The molybdenum mining industry and modern use of the metal originated in Colorado with the Climax mine. Mining claims were staked on Bartlett Mountain in 1890 by Sam and John Webber, and E. G. Heckendorf. At first they thought they had a silver mine, but the mineral was later identified as containing molybdenum sulfide. By 1911 Heckendorf began consolidating his control over the mountain. The Climax Molybdenum Company was incorporated in 1916.

Initially the company found little demand for their commodity, therefore, they conducted research to identify uses for the silver colored metal. The principle use is as an additive to steel to impart hardness, strength and corrosion resistance. Its chemical applications include reagents, catalysts, pigments, and lubricants.

The mine found an important market during World War I as an alloying agent for steel. Output from the mine was around 250 tons per day of ore. The block caving method of mining was installed in 1935 and production soon rose to 18,000 tons per day.

In block caving a large horizontal area is undermined, then in a single blast all of the rock supports are blasted away leaving a 200 by 300 ft. (61 m by 91 m) area with out any support. Gravity causes the rock to break apart and come caving down into a network of funnel shaped mine openings that are used to feed the ore to an underground train system. As more ore is taken away, the cave progresses upwards into the mountain. Over the years problems have been encountered with the block caving method of mining because of variations in the strength of the rock (King, 1945). Rock mechanics investigations were undertaken to solve the problems of excessive stress closing mine openings, arching of the caved rock, and the formation of funnels through the orebody causing dilution of the ore by overlying waste rock. The arching problem was solved with a number of record blasts. One of the big blasts was on October 2, 1955 when 81,614 pounds (37100 kg) of explosives were detonated in a single blast to break 750,000 tons of ore. The largest blast of all was in 1964 when 416,000 pounds (189,100 kg) of explosives broke 1.5 million tons of ore defining the gloryhole depression over the caved ground (Cappa, 2001). Funneling was solved by carefully monitoring the rate of extraction of ore from any given draw point. Attempts to stop the closing of mine openings caused by intense pressure included heavy steel supports and reinforced concrete but they were only temporary fixes. Instead, the company learned to make due by re-excavating the openings on a periodic basis as needed. Elevations at the mine range from 11,000 to 12,000 ft. (3,354 to 3,658 m). Winter operations were hampered by freezing temperatures and large amounts of snow. In the past, the spring snowmelt disrupted operations when water surged into the underground workings.

The company remains an industry leader. Climax held the title as the world's largest underground mine for many decades. Tailings disposal required the construction of a system of impoundments which have covered the former town of Kokomo. Other mines were developed. The Urad mine was opened near the town of Empire in the late 1950's and produced until 1974. Open pit mining was initiated at Climax in 1973 while the block caving operation continued. Production at Climax rose to 50,000 tons per day in the 1970's and the workforce grew to 3,000 people. As of January 1, 1974, 312,536,614 tons of ore had been extracted from the Climax deposit (Climax Molybdenum Company, 1974). The mine suspended operations in 1986, however, significant reserves are left in the open pit. Formation of acid rock drainage at the mine and in the tailings is another challenge facing the company. They are addressing the problem with a multi-year reclamation program. They are amending the surface of mine wastes with limestone and organic matter and seeding to establish vegetation. Water is collected and treated to ensure there are no acid discharges from the site.

The Climax geologists, puzzled by the placement of the Urad ore body eventually found another large deposit. The Henderson mine was opened as a result in 1976 after an investment of $500 million. It was Colorado's largest privately financed construction project and it currently produces between 20 and 25 million pounds (9 and11 million kg) of molybdenum sulfide per year. The Henderson mine includes a 9.6-mile-long (15.4 Km) haulage tunnel that carries the ore across the Continental Divide to a modern milling facility. In the year 2000, the Henderson mine received a $100 million upgrade. The rail haulage tunnel was enlarged for installation of a conveyor system and the mill was expanded. Colorado remains the leader in molybdenum production.

Oil Shale

Holding an estimated two trillion barrels of oil, portions of the Green River Formation contain a petroleum-like organic material called kerogen. Upon heating an oily liquid is released from the shale. The Mahogany zone is a 100-ft.-thick (30.5 m) section of oil shale that outcrops along Parachute Creek in northwest Colorado that yields more than 25 gallons per ton. First described in government reports in 1914, there have been three rushes to the area (Scamehorn, 2002). The first was in 1916 in response to increased petroleum prices. By 1920 over 100 companies incorporated with plans to mine and process Colorado oil shale. A few years later the discovery of the East Texas oil fields drove the price of petroleum down making oil shale production an unattractive venture. Only the federally funded operation at Rulison, Colorado continued to operate until 1928.

World War II caused another petroleum shortage. In 1944 the Bureau of Mines was called upon to again research the development of alternative fuels. The Anvil Points plant was constructed seven miles west of Rifle and operations ran from 1947 until 1955. Sinclair Oil tested in-situ retorting in 1953 and 1954. Union Oil Company constructed a processing plant that opened in 1957 and processed 1,000 tons per day of shale. Lower oil prices again ended oil shale activities.

Although there was some commercial activity in the 1960's, the most recent oil shale boom was in response to the Arab oil embargo of 1973. Over two billion dollars was spent purchasing leases and constructing towns, roads, utilities, mines, and processing plants. Exxon's $400 million Colony project was the most advanced. It would have extracted and processed 66,000 tons per day. The environmental implications required studies of all aspects of the operation including health hazards from shale dust and fumes (Coomers, 1981). The implications for water quality and land disposal were also evaluated. At the peak of the activity, the oil shale industry was considering disposal of almost 100 million tons per year of spent shale, 20 million tons from the Colony project alone. Several projects were nearing commercial production when oil prices declined. Oil shale activities were suspended in the early 1980's. It is now a matter of when rather than if an oil shale industry can be developed in Colorado.

Coal

Coal was discovered in 1859 about 14 mi. (22.5 Km) north of Golden. The outcrop was traced south and men began gathering coal for heating that first year of the gold rush. Coal fueled the smelters, railroads, steel mills, and factories of Colorado. The state has produced over one billion tons of coal since 1864 from over 1,700 coal mines. It has been Colorado's most valuable commodity. At present coal-bed methane, natural gas, oil, and carbon dioxide production contribute more value than coal.

In the 1970's coal mining came under increasing regulation culminating with the Federal Surface Coal Mining and Reclamation Act in 1977. This law regulated coal disturbance and reclamation requirements. It required detailed mine planning and stipulated minimum standards for waste disposal, sediment control, topsoil salvaging, and revegetation. Surface mines were required to backfill open cut highwalls, re-grade waste banks within 180 days of disturbance, and plant native plant species on replaced topsoil. The law triggered millions of dollars worth of scientific and engineering investigations into all aspects of mine reclamation. The studies include rock and soils mechanics investigations to limit the environmental effects of subsidence and ensure the stability of dams and surface mine waste embankments.

Colorado's coal reserves are estimated at approximately 16 billion tons. In 1999 its output of 30 million tons was ranked 11th among the 30 coal producing states. Technology and innovation continues to bolster Colorado coal. In June 1997 The Twentymile Coal Company's Foidel Creek mine set a new monthly world production record when longwall technology helped produce 1,100,401 tons of coal. Two of the state's mines the West Elk and the Foidel Creek continue to battle for the title of the most productive longwall system as the record continues to be reset higher still.

INNUNDATION

Keeping water out of the mines has been a significant challenge. Many mines have been flooded when faults and other water bearing formations have been intercepted by underground workings. There have been a few disasters. In 1889 the White Ash coal mine at Golden drove a tunnel under Clear Creek. The tunnel collapsed and the resulting inrush of water killed 10 men working in the mine.

On August 29, 1895, water in the old Fisk mine near Central City broke into the adjacent Americus mine killing two, another 12 died when the water continued its rampage breaking into the Sleepy Hollow mine. In response to these events, mine maps are placed in archive with the State of Colorado for future use. Mines operating near old workings and bodies of water are required to carefully plan the work and probe the ground ahead with long drill holes that would allow time to escape should large bodies of water be encountered. Where possible the old workings should be drained prior to excavating below the water filled passageways.

History nearly repeated itself nearly in 1979. The Sunnyside mine near Silverton followed high-grade gold ore extending the mine workings under Lake Emma. Some exceptional specimens of crystallized and wire gold were recovered as the mining operation progressed upwards towards the bottom of the lake. On a Sunday in December the lake broke through collapsing into the C level of the mine. The resulting flood destroyed miles of mine workings and shot out of the mine entrance filling the receiving streams with a torrent of muddy water. The lake is gone. Fortunately none of the 150 miners were underground at the time of the disaster.

SOLVING ENVIRONMENTAL PROBLEMS

Inactive and Abandoned Mines

Colorado, like other states with a legacy of mining, has a significant number of inactive and abandoned mines. Although estimates vary, the State is believed to have more than 20,000 underground mine openings. Water quality has been degraded in hundreds of miles of streams and rivers from drainage from mine openings and by storm water runoff passing over mine waste rock and tailings discarded with little consideration of the potential environmental consequences.

Passage of Federal environmental regulations such as the Clean Water Act of 1972 and the Surface Mining Control and Reclamation Act of 1977 and their subsequent implementation has resulted in a significant reversal of the negative impacts from mining. Although numerous sites remain to be addressed, many major mine reclamation projects have been completed. Mine reclamation projects in the State of Colorado include removal of mine waste rock and tailings from streams and wetlands, diversion of clean water away from disturbed areas and mine waste materials, and construction of water treatment plants to mitigate the worst mine drainage sites. The State Division of Minerals and Geology has implemented an aggressive cleanup program that has closed several thousand dangerous mine openings and has reclaimed hundreds of mine sites. Mine water treatment plants in Colorado include the Bureau of Reclamation's facility at the Leadville Drainage Tunnel, the State of Colorado's facility at the Argo Tunnel in Idaho

Springs, the Environmental Protection Agency's facility at Summitville, and several smaller plants operated by private industry. Most of these plants are expected to be required in perpetuity since the source rock is predicted to continue to generate acidic and metal laden water for hundreds to thousands of years.

Modern Mine Closure and Reclamation

Mine closure is an evolving art. In Colorado it has evolved into a complex applied science. The process begins with site characterization to understand the nature of the problems to be solved, the availability of material resources that can be used in reclamation, and identifies the environmental and cultural resources to be protected. The object is to achieve a stable landform that maximizes future land use with no offsite impacts.

Around the world, wet, high-altitude sites with acid rock drainage are the most challenging mine sites to close and reclaim. In addition to the standard mine reclamation procedures such as closing mine openings, diverting water, removing wastes from waterways, and revegetating disturbed lands with native vegetation, these sites require the development of new techniques. Encapsulation of mine wastes in engineered repositories, installation of bulkheads to control drainage from underground mines, and development of innovative passive and active treatment technologies are procedures that are being applied to difficult sites such as the Summitville mine in southern Colorado (Campbell and Gobla, 2000). Difficult reclamation problems remain. Mitigating degraded runoff from steep mine highwalls, managing water quality in pit lakes, and eliminating acid drainage from underground workings are difficult mine reclamation problems that require more innovation, development, and experience to solve.

In the 1980's principles of geomorphology, soils, and vegetation science were researched and developed by the surface coal mining industry. This work greatly advanced surface coal mine reclamation and has been transferred to the other mining sectors. Environmental protection issues focused on protecting threatened and endangered species, saving topsoil, preventing off site sedimentation, and restoring vegetation that would benefit big game and other wildlife. Trying to look ahead is some times best served by looking backwards.

In the 1990's more attention was placed on metal mine closure. Better closure techniques for open mine shafts and adits emerged enhancing public safety. Many of the techniques for closing mine openings were developed in Colorado (Dolzani and others, 1995). The application of geosynthetic materials advanced into the development of waste repositories. A wider vision of environmental protection emerged in the form of considering impacts on bats, stream biota, and the problems from invasive species.

Major mistakes in metal mine environmental management and reclamation made in previous years began to emerge around the nation. The most prominent example was the abandonment of the Summitville gold mine in Colorado in 1992 (Posey, Pendleton, and Van Zyl, 1995). The poor understanding of geochemistry, combined with extreme climatic conditions at a site above 11,000 ft. (3,350 m) in elevation, resulted in hard lessons for the industry and regulatory community. The government had to step in and take over the bankrupt mine with severe acid rock drainage problems. The site required $170 million to close, 60 percent of this was spent on water treatment. In the aftermath of Summitville, metal mining is under intense regulation. It led to development of better site characterization and advanced the understanding of the long-term chemical behavior of mine wastes. Mining can only proceed if it can operate and reclaim the mine without impacting receiving streams. This is a difficult challenge and some wondered if another gold mine could ever open in Colorado in the aftermath of Summitville. The Cresson mine at Cripple Creek has met the challenge with sound engineering principles and proactive management. This open pit mine has already produced over one million ounces of gold and is working on extracting reserves of five million ounces (O'Neil, 2001).

The current decade brings the emergence of new challenges and new solutions. Identifying ecological risks and cumulative impacts are new environmental issues. Neutral rock drainage, water that is not acidic but contains high levels of metals or salts, is perhaps the most important technical challenge to evolve in mining in the last few years. Although this less common problem was identified at several sites many years ago, it became a significant issue after the Maybe Canyon phosphate mine in Idaho began discharging large amounts of selenium from a "clean" waste-rock dump into a mountain stream. Neutral rock drainage is now an additional issue to be considered as part of mine reclamation planning. Microencapsulation and organic amendment to initiate sulfate reduction are emerging innovations to deal with acid rock drainage and may help with neutral rock drainage as well.

Looking back suggests that as time goes by, more innovations will emerge. Mine reclamation has progressed from shaping, covering waste with topsoil, and revegetation in the 1980's, to secure disposal of mine waste in repositories in the 1990's, and is now moving into preventing impacts by chemical treatments of mine waste rock and spent ore. These emerging technologies will be developed in the coming years and Colorado is playing a leading role. Mine reclamation will become more efficient, effective, and more complex as time passes.

SUMMARY AND CONCLUSION

Colorado has a rich legacy of mining which has faced and overcome many challenging problems. Although millions are spent every year on treating water and cleaning up problems from past mining sites, the State's mines produce around one billion dollars of products each year. The Colorado mineral industry is diverse, continues to innovate to overcome challenging issues, and shares its advancements in mineral production, environmental protection, and reclamation technologies with the rest of the world.

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COAL MINE SUBSIDENCE—WHAT IS THE RISK TO DEVELOPMENT?

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Key Terms: coal mine, subsidence, mine shaft, room and pillar, longwall, sinkhole

ABSTRACT

There are hundreds of abandoned coal mines throughout Colorado that present varying degrees of hazards due to encroaching development as the state's population grows. Among these hazards are the potential for subsidence over mines, sinkhole development, and mine shafts that have not been appropriately closed. Early mines were worked by room and pillar mining, where 5 to 40-ft wide (1.5-12.2 m) coal pillars were used as support. Often, as an area was worked out, the pillars themselves would be mined in retreat mining. Typically, subsidence of the mine roof and overlying strata occurred over extracted areas soon after mining operations ceased. However, this is not always the case; in particular, shallow mines (less than 100 ft (30.5 m) deep) can continue to present problems as fluctuating ground water causes changes in the overlying soil and bedrock and also causes the remaining coal pillars to slake.

Subsidence becomes a hazard when the effects are transferred to the surface. Sagging ground can cause distress to structures, pavement, and utility lines. Structural damage and disruptions to infrastructure can also occur as a result of the rapid and unexpected development of sinkholes above abandoned mines. These features, though relatively rare in occurrence, are a result of shallow workings beneath weak bedrock and /or thick soil. Surface ruptures and open mine shafts can present a safety hazard.

Because of discrepancies in surveying and inaccuracies in maps (both intentional and unintentional), identification of problem areas is not always straightforward. Some mines were not mapped at all. Also, multi-level mining, minor shafts or unrecorded workings might not be indicated on mine maps. Subsidence investigations for proposed development should aim to confirm the accuracy of the mine map and determine the extent of subsidence through drilling and associated geophysical logging. Mine maps are available from the Colorado Geological Survey. The issues involved with land use planning should be carefully evaluated when considering undermined areas. Most undermined areas are suitable for development, but some require mitigation or use restrictions.

INTRODUCTION

Coal mine subsidence is the process where roof rock and unconsolidated material overlying an underground coal seam caves into the resultant void space after extraction. The process may affect both natural and manmade features at the ground surface, depending on the local geology and conditions of the mine.

The Colorado Division of Minerals and Geology has calculated that there are about 50,000 acres in the state underlain by abandoned coal mines (Figure. 1).

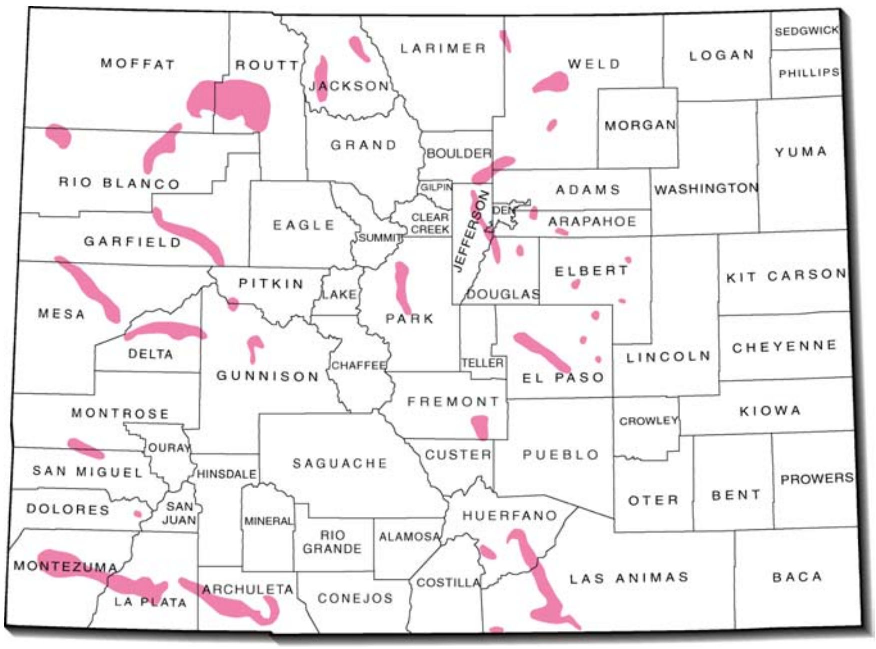

Figure 1. General locations of inactive coal mines in Colorado.

In 1985 the Colorado Geological Survey estimated there were 13,000 people and 5,000 houses on undermined areas along the Front Range (Turney, 1985), an area which contains no active coal mining. Boulder County can be used as an example of an area where the number of people potentially impacted by undermining has increased. Since 1990, the population of Boulder County has grown by 30 percent, with much of the development occurring in the Boulder-Denver corridor that is coincident with coal mine locations. A detailed evaluation of Boulder County using year 2000 census figures shows that about 13,500 people now live over abandoned mines. In Colorado, there are 1724 abandoned coal mines, including 405 mines recorded for the Denver Basin coal region (Carroll and Bauer, 2002). Although many mines are located in remote areas, the number of people statewide living over former coal mines will increase in the future. It is worthwhile to consider the potential subsidence risks to property damage and human safety.

MINING METHODS

Coal mining began in Colorado in the 1860s, after gold prospectors found near-surface seams that could be exploited. Room and pillar mining, which was the method used almost exclusively in early coal mining, allowed 40 to 70 percent of the coal to be removed. The mine was initiated with a shaft or slope driven or dug to the layer of coal. Then, two or more parallel entries or haulageways were excavated in the coal seam for the purpose of removal and ventilation. Openings or rooms of coal were excavated on either side of the main entries. The rooms and the haulageways were supported by pillars of coal left in place to brace the roof of the mine. Timber mine props were also used in older mines. Mine maps show that the rooms narrow toward the haulageway, with associated widening of pillars for extra support in this area. Figure 2 shows a plan view from a typical room and pillar mine. The orientation of rooms might be the result of the mine geology and faulting that was encountered, or might have been the decision of the mine superintendent to take advantage of drainage, cutting, or gas removal.

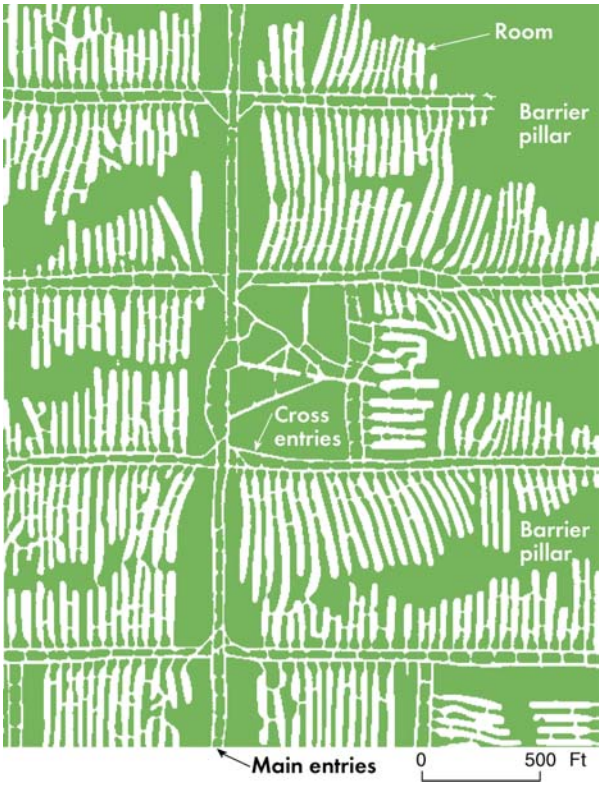

Figure 2. Possible layout of an early room and pillar mine, showing irregular room and pillar sizes.

Large barrier pillars, sometimes ranging to 150 ft (45.7 m) in width, were commonly left in place to support a surface feature such as a railroad track, roadway, or stream. This type of planning showed that early miners had an understanding of surface subsidence.

Extraction panels for retreat mining were designed somewhat differently from the above examples, and when mining of an area was completed, pillars were pulled. Theoretically pillars could be pulled until the roof started to cave; partial pillars might have been left, and these are often indicated as such on maps. "Pillar robbing", which probably began in the 1920s, was an inexact science that was accompanied by inexact reporting. One of the objectives of mine maps was documentation of coal extraction for fiscal purposes, including royalty and tax. In some areas, pillars were mined, but not reported as such.

Shortwall mining utilizing a continuous mining machine replaced room and pillar mining in Colorado beginning in 1946. This method continued the use of the room and pillar mine plan, but room size was increased. Due to the substantial increase in mining rates and the symmetrical room cross-sections, a greater distance between supports was possible, resulting in higher extraction ratios and theoretically more thorough subsidence.

SUBSIDENCE MECHANISM AND FEATURES

Mine subsidence begins when roof support is withdrawn and overlying rock, and possibly unconsolidated sediment, collapses into the resulting void space. Longwall mining, which is the mining method used in most modern underground mines and which is characterized by mechanized mining methods and uniform panels, produces more predictable subsidence than room and pillar mining. The strength of the roof rock is an important factor in predicting subsidence. In longwall mines, as the working face advances, the roof rock sags into the mine void behind a cutting machine called a shear. The sag is known as *trough* subsidence, and is essentially complete within a short time after mining of an area has ceased. Ground strains transmitted to the surface can cause cracking and bending in structures, depending on whether tensional or compressional stresses are involved (Figure 3).

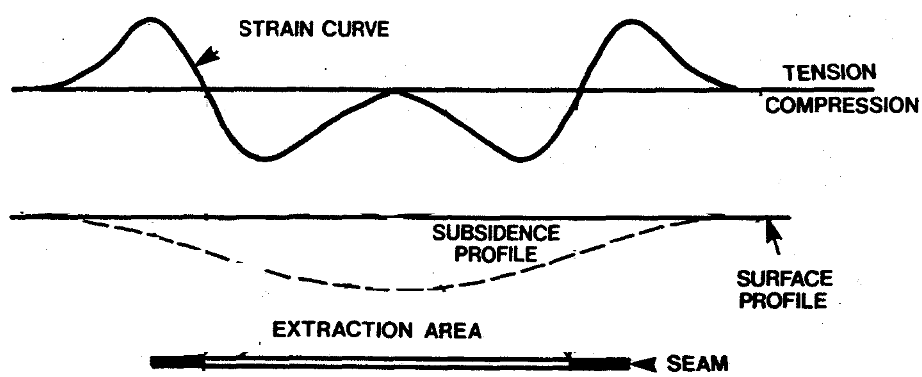

Figure 3. Subsidence and strain related to mining.

In contrast, subsidence related to room and pillar mining might be associated with intersection failure (a large amount of void space; intersection failure is generally a problem in shallow mines) or pillar failure, and is not predictable partly because of the variety in pillar sizes and the uncertainty of whether pillars remain. Pillars can fail years after mining has ceased depending on whether there is void space available into which they can crush. Pillars can also punch through a soft mine floor, with subsequent sag of the overlying roof rock. A possible factor in decreasing pillar strength is the oxidation of coal that is exposed to air, or fluctuations in ground water from seasonal variations or changes related to drought cycles. Ground water has the combined effect of protecting the coal from oxidation and providing a buoyancy that reduces the load on pillars.

Subsidence associated with room and pillar mines can produce tensile and compressive ground strains and damage similar to that from longwall mining. Trough subsidence, which is the primary subsidence mechanism in longwall mining, can occur in room and pillar mines where a large panel of coal has been mined, or with the progressive sudden failure of adjacent pillars. As the roof sags into the void, the ground surface sags correspondingly and exhibits tension cracks near the perimeter of the feature, and ground compression in the center of the trough (Figure 3). Figure 4 shows a manifestation of compressive ground strain. Site specific geologic conditions, including depth of mining, pillar size, and roof and floor rock lithology, greatly influence the time dependent component of subsidence. Both empirical observations (Sherman, 1986) and geotechnical analysis (Matheson and Bliss, 1986) indicate that collapse is both more rapid and complete when low compression and tensile strength rock occurs in the mine roof and floor.

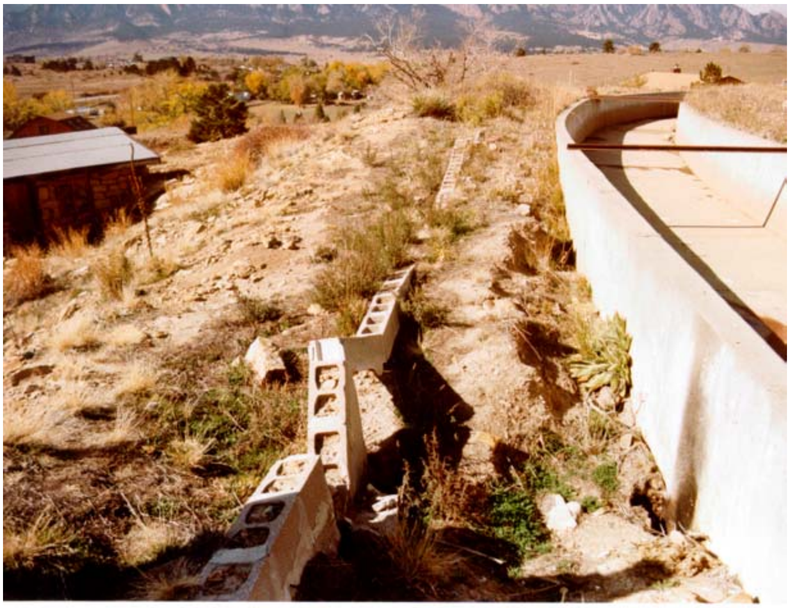

Figure 4. Compression associated with subsidence has upended the cinder block wall. Marshall area, Boulder County.

Chimney subsidence is a feature of room and pillar mining and occurs as progressive caving of overlying material allows voids to propagate upward, and is related to depth, overburden, and span support. The size of the surface features, manifested as pits and sinkholes, is generally restricted to the width of the mine rooms or mine entries. (Notable exceptions exist over the Klondike Mine and the McFerren shaft in Colorado Springs). Chimney subsidence is most likely to occur over inadequately sealed shafts or slopes and where there are three- and four-way entry and room access intersections that could produce a conical collapse geometry and also allow more space underground into which the roof rock can collapse (Abel, 2001).

In dipping coal beds that have been mined, such as those along the foothills of the Front Range and in the Piceance Basin and near Newcastle, voids can migrate vertically and diagonally upward (updip) along the bedding surface, a process called stoping (Turney, 1985).

Mine shafts present special problems. The shaft might represent open or discontinuously open space from the surface to the mined horizon. Shafts, including hoist shafts and ventilation shafts, are typically indicated on mine maps, but not always. Shafts might have been inexpertly closed by filling the void with debris, which can shift at some time in the future. Although small in area compared to an overall mine plan, shafts that have not been structurally sealed represent the greatest subsidence hazard (Figure 5).

Figure 5. Subsidence related to shaft opening in a trailer park.

In shallow mines the possibility of surface expression from both chimney and trough subsidence increases where there might be insufficient overburden to fill the void space before the ground surface is breached. (This relates to the bulking of the overburden, discussed in the next section.) Surface expressions of mining can be seen in the Marshall area in Boulder County (Figure 6) and near the Erie cemetery in Weld County. At both locations, where mining occurred at depths of 50 to 100 ft (15.2 to 30.5 m), the surface outlines of rooms and pillars can be delineated. Sinkholes and pits are also present (Figures 7 and 8). West of Frederick over the Shamrock mine, present at depths of about 67 ft (20.4 m), sinkholes appeared in September 1994, about 40 yrs after the mine was closed (Figure 9). In Colorado Springs, Dames and Moore (1985) mapped sinkholes associated with shallow mining (40 to 75 ft (12.2 to 22.9 m) deep) in the Cragmor/Country Club part of the city. Thorburn and Reid (1978) documented a case of trough subsidence that occurred more than 100 yrs after mining had ceased over mine workings located about 50 ft (15.2 m) below the ground surface.

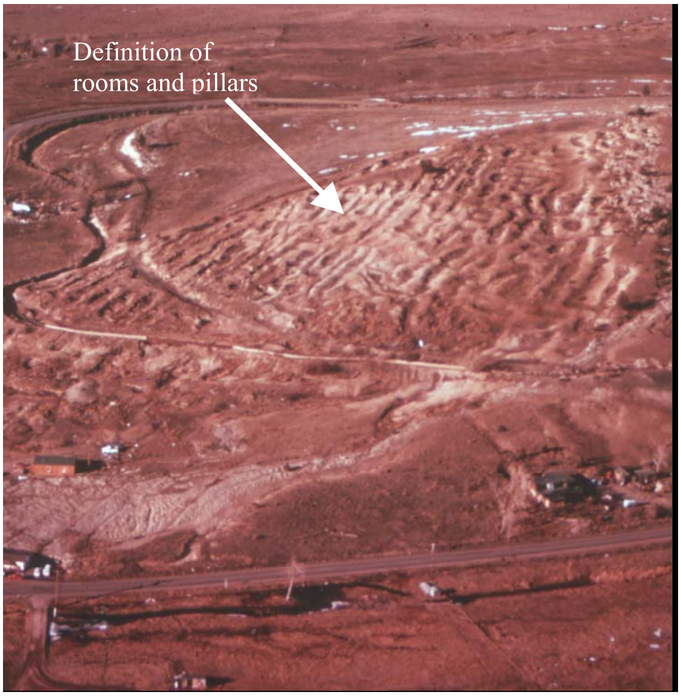

Figure 6. Subsidence near Marshall, Boulder County.

Figure 7. Sinkholes in the Marshall area.

Figure 8. One of several subsidence pits northwest of the Erie cemetery.

Figure 9. Sinkhole associated with the Shamrock mine, west of Frederick, Weld County.

EVALUATING SUBSIDENCE POTENTIAL, THE UNCERTAINTY FACTOR

As with any geological hazard, the steps toward mitigation of potential risks associated with coal mine subsidence begin with identification of the problem.   Subsidence maps, showing the extent of mining, are available for all of the coal basins of the state. In coal fields correlative with populated areas, notably the Boulder-Weld coalfield and the coalfields of Colorado Springs, subsidence investigations have been performed that have included a hazard ranking (Dames & Moore, 1985, 1986; Myers et al., 1975).

In viewing maps of mines that have been closed for 70+ yrs, consideration must be given to whether the datum point for a map remains valid in light of more recent surveying of the state. Mine maps might be inaccurate because of surveying errors (where a datum is mislocated) or because of because of deliberate misreporting for fiscal purposes. The importance of the accuracy of mine maps was well illustrated in July 2002. Mining at the Quecreek mine in western Pennsylvania breached the wall of a neighboring inactive mine where the location of workings was not definitive. Impounded water flooded the mine, trapping nine miners. Mis-siting of a survey datum has similarly resulted in the mislocation of several mines west of Erie, Colorado that were active in the late 1800s.

One piece of critical information to obtain in predicting future subsidence is how much void space remains in a mine, and conversely, how much subsidence has already occurred. The statement that often appears in reports, "authorities agree that most subsidence occurs almost immediately after mining ceases" (proof by assertion) is not a substitute for a study. However, it is not possible to determine present mine conditions for the entire extent of an abandoned mine. Abandoned mines are generally filled with water and are difficult to enter. A drilling program will provide data from selected points. However, if a boring is targeted to intersect a pillar and encounters mine workings, it might be difficult to determine whether the findings are due to inaccuracy in locating the boring, inaccuracy in projecting the boring to the mine horizon, or are due to a pillar that has failed or been mined. As geophysical techniques improve, they might be used to provide a more comprehensive evaluation of subsurface conditions.

In a boring operation, data can be obtained that allows the geologist or engineer to make assumptions about the subsurface conditions within a certain area or within a radius of influence. Void space can be estimated by changes in drilling rate. Potential rubble zones of collapsed roof rock can be identified by chatter of the drill string and loss of circulation. Geophysical logging should follow drilling operations to provide additional information, although some operators are hesitant to lower tools into a zone that has caved. A caliper tool is designed to record the size of the borehole opening, making it possible to identify rubble zones and void spaces. In most cases where a caliper tool is run, however, there is incomplete extension of the caliper arms, indicating a partial void that is probably associated with collapse. The material around these voids might be subject to future movement. Video cameras have some use in evaluating workings in open passages (Kirchner and Colaizzi, 1986) but are of less value where the field of observation consists chiefly of broken rock.

For shallow mines where subsidence effects can be seen at the surface, it is possible to compare a detailed topographic map of the site with the mine map to first determine how accurately the mine is located, and then to ascertain what pillars might be remaining and perform a pillar stability analysis. If a sinkhole is observed, it cannot be assumed that subsidence is complete. CGS has received a number of reports from landowners in Weld County who fill holes on a continual basis.

The uncertainty involved in predicting subsidence over room and pillar coal mines, can be reduced by in-depth study of a mine or coalfield. Dames & Moore (1985, 1986) performed such investigations for the mines in Boulder County and in the Colorado Springs area. Data from field exploration and laboratory testing were used to statistically evaluate areas and assign a risk value for future subsidence. The study concluded that the appearance of sinkholes at the ground surface is primarily a function of the depth to the mined horizon, the thickness of the mined horizon (mining height), and the strength and bulking characteristics of the overburden material. Furthermore, Matheson and Bliss (1986) used the same data to project that chimney subsidence in relatively flat-dipping rocks along the Front Range can be expected to occur most frequently in an overburden thickness/mined height ratio of less than 10 in room and pillar mining areas, and 15 in areas where pillars were pulled (which increases the extraction width or the span to depth ratio). Most areas underlain by abandoned coal mines have not been evaluated in this kind of detail, although subsidence investigations have been performed for individual properties as a precursor to development.

With an appreciation for the difficulty in predicting subsidence over room and pillar mines, Piggott and Eynon (1978) theoretically calculated the critical depth for mining below which subsidence would not be manifested at the surface. For chimney subsidence this formula is

$$\frac{3h\ (100)}{\%S}$$

where h is the thickness of the seam and S is the amount of bulking or increase in volume that occurs when the roof rock fills the cavity. S is calculated as

$$\frac{Vc\text{-}Vo}{Vo}$$

where Vc is the volume of the collapsed roof rock and Vo is the original volume of unbroken strata. This formula shows that the less competent the roof rock, the greater the vertical distance that subsidence will progress, i.e., the more material necessary to fill a void. (Bulking can only occur when rocks bridge voids.) The formula is difficult to accurately apply to field situations unless the increase in volume is known, rather than estimated or projected from an alternate location. Also, near-surface conditions are generally more variable than at depth. The volume of collapsed overburden material may reach an equilibrium but might be later destabilized by fluctuations in ground water and shifting or flushing of sediments. The rule of thumb that a surface void will not be deeper than the thickness of the extracted zone is invalid if the void space is filled with unconsolidated sediments or weak friable rock. The use of these formulas has gained favor as it permits conclusions without performing site-specific investigative work.

The National Coal Board of Great Britain (NCB) developed a model for calculating subsidence over longwall mines based on empirical observations (NCB, 1979). The model uses mining height, depth to mining and panel width, and the number of seams that were mined, as input parameters to determine the amount of subsidence or strain that would be transmitted to the surface. The Office of Surface Mining (1993) produced a similar model for subsidence prediction for both longwall and room and pillar mining in eastern U.S. mines, adding percent hard rock and extraction ratio as input parameters (OSM, 1993). Both models might be applied to room and pillar mining where mine maps are judged to be reliable. For prediction of future subsidence, the input for mining height could be an estimate (based on drilling) of the remaining void space. Neither the NCB nor the OSM model allows input for mines shallower than 100 ft (30.48 m) deep. The models allow the user to obtain worst case horizontal and vertical displacements; structures can then be designed accordingly.

Where pillars are known to remain, the stresses on pillars can be calculated and a factor of safety can be assigned (OSM, 1993). Numerical models for subsidence prediction exist where iteratively, a pillar is caused to fail and the load is redistributed to remaining pillars. A worst case scenario involving trough subsidence can be predicted based on failure of pillars. Maximum subsidence can be used to calculate ground strains. Roadways and structures can then be designed accordingly. This method of evaluation requires minimal or no site-specific investigative work.

In Colorado, the OSM is charged with responding to emergency mine-related situations (those that pose a risk to safety or property). The majority of such cases involve shafts and slopes that have not been closed or that have been closed inadequately. It is critical that these features be located and evaluated before development begins. However, where shafts and slopes are not identified on mine maps, the initial recognition might be when an opening appears in a developed property or roadway.

DATA RESOURCES FOR EVALUATION

CGS operates a Subsidence Information Center (SIC), where maps showing the extent of mining in all of the coal basins within the state can be obtained. As stated above, detailed studies are available for certain coalfields in the state (Myers et al., 1975; Dames and Moore, 1985, 1986). The SIC library is also the repository for maps of the 1724 coal mines in the state and for numerous site-specific reports. The following factors should be considered when viewing mine maps: mines may be known by more than one name; without a datum, it is difficult to precisely locate a mine; mining might extend into the area of another mine; a mine map will not necessarily say whether two seams were mined in an area; generally, the most accurate and complete map in a series is one prepared and submitted the year that mining ceased.

The Colorado Division of Minerals and Geology maintains a database that includes mine shafts that have been sealed. Shaft locations are demarcated in the field with a brass marker.

PLANNING AND MITIGATION

Avoidance of undermined areas, as with all geologic hazards, is the surest means of eliminating risk, but this is not always practical or desirable. Also, prohibiting development of a property can deny what the landowner perceives as his right to develop a site. Counties and municipalities must balance the issue of property rights with the concerns for safety of future residents and structures. In high risk areas, local governments must also consider potential costs of repair to infrastructure, the impact of damaged commercial property, the loss of bank financing, a drop in real estate prices, and possible legal consequences.

Except for undermined land that has been developed historically as part of a town, city, or county, most of the current undeveloped undermined parcels are agricultural land that require a change in zoning (where zoning exists) for conversion to residential or commercial development. It is at this juncture, or at the sketch plan stage (when use and density are decided), that governments should raise questions or request information on the suitability of a site for the proposed use. A number of local governments have master plans or overlay zones, which mandate site-specific investigations as a requirement for development where conditions are known or expected to be problematic. More often than not, individual homeowners are not equipped to evaluate the implications of undermining. Even with disclosure of site conditions, it is generally the case that if a property has been approved for development, people do not expect any problems.

Most undermined properties can be developed providing the hazard has been identified and the risk has been adequately addressed. Modifications to a site plan might be as painless as establishing building envelopes, reorienting structures toward positions of least ground strain, and designing structures and roadways to span the subsidence features that might be expected at a site. In southern Jefferson County, Coal Mine Road was constructed to span the diameter of pits that had occurred over the Economy Mine. Foundations can be designed to resist forces associated with subsidence settlement or heave. The Department of Housing and Urban Development recommended specially designed raft foundations that could protect the buildings from damaging horizontal strains and distortions; strengthening buildings to resist anticipated horizontal strains and distortions; or making buildings flexible enough to accommodate anticipated horizontal strains and distortions (Yokel et al., 1981). Flexible utility connections, which are fairly standard in current construction, would also limit damage.

Development pressure in the future may make it economic to construct buildings with deep foundations that bypass mined zones, or to fill subsurface voids with cement slurry. The current cost of both these alternatives is extremely site specific. However, approximate costs range from $1.50 to over $7.00 per square foot.for grouting in a collapsed mine with 20 percent void space.

Where shafts are known to be present on a property slated for development, the method of closure should be confirmed. Shafts might be adequately closed for general safety purposes, but could be structurally unsound to support roadways or buildings.

Government agencies charged with oversight of abandoned coal mines do not concede that a safe depth exists below which mining would not pose a subsidence hazard (Harrison et al., 2003, personal communication). Although, surface manifestations (sinkholes, pits, troughs) are almost always the result of subsidence over mines less than 100 ft (30.48 m) deep, the local geology can change this premise (such as where a thick sequence of glacial till is present). Where mine operations are active, the Pennsylvania Department of Mines and Mineral Industries did not allow mining where the overburden was less than 100 ft (30.48 m) (Cortis, 1969), indicating this to be an unsafe depth. Public or common open space is a solution that has been suggested for areas where subsidence might breach the surface, but such locations could pose a risk to human safety. In southern Jefferson County in the vicinity of the high-dip Littleton Mine, OSM plugged a deep shaft that subsided in the open space. A daily check by open space maintenance personnel is prudent to determine whether pits have developed. The use of geotextile fabrics to support play areas is a possibility, but the strength of these materials to mitigate subsidence conditions should be confirmed.

In 1977 Congress enacted the Surface Mining Control and Reclamation Act. Title IV of this act allowed a trust fund to be established to address problems associated with abandoned mines, including subsidence (30 U.S.C 1231, 1232, 1253). The federal administrator of the fund is OSM, which allocates monies to the Colorado Division of Minerals and Geology Mined Land Reclamation Board to support the Mine Subsidence Protection Program (MSPP). To qualify for coverage by MSPP, a residence must have been built before February 1989, by which time it was judged that information on abandoned coal mines and their locations was fairly well known, and that subsequent homes had been constructed with an understanding of the risks. (Where maps of an area have been shown to be inaccurate, owners of post-1989 homes can petition the Board for inclusion in MSPP). Claims for damages attributed to coal mine subsidence are investigated by a geotechnical engineering firm contracted by Marsh, Inc. the insurance company associated with the program.

SUMMARY AND CONCLUSIONS

In summary, if a site is to be developed, the uncertainty about subsurface conditions must be minimized. A qualified geologist or mining engineer with subsidence experience should evaluate the property. Drilling should be performed to confirm the accuracy of the mine map (the extent of mining and the mine plan, itself), obtain preliminary information on remaining void space, and to formulate a contingency plan for further work based on the initial observations. In an area where mine maps have been shown to be inaccurate, drilling should be performed beyond the known extent of mining. On a property that exhibits surface impacts, the strength of the roof rock and the potential for future pillar collapse, as well as remaining void space, are important criteria in determining what structural design modifications might be necessary. Nationwide, state agencies that deal with abandoned coal mines do not consider that there is a maximum depth below which mining might not be considered a problem. Mines less than 100 ft (30.48 m) deep have been shown to continue to subside decades after mining has ceased. In such cases, it is not possible to determine when subsidence will occur.

ACKNOWLEDGEMENTS

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GEOLOGIC CONTROLS ON COMPLEX SLOPE DISPLACEMENT AT THE PITCH RECLAMATION PROJECT

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Key Terms: slope stability, regressive slope, mine reclamation

ABSTRACT

Moment-driven, regressive slope deformation has been observed in many large open pits. The Homestake Pitch Mine provides another example of this type of pit slope instability. The North Pit of the Pitch Mine was developed in a geologic setting that led to moment-driven slope instability. Moment-driven slope movement is typically regressive and often manageable during mining operations. However, pit excavation, coupled with extreme climatic conditions, led to large-scale, rapid slope failures that eventually terminated mining in the North Pit. The Pitch site is currently in reclamation. The North Pit slopes are in a regressive state and displacement velocities continue to decline.

INTRODUCTION

The Homestake Mining Company (Homestake) Pitch Reclamation Project (Pitch) is located in the Sawatch Mountains, approximately 30 mi (19 km) east of Sargents, Colorado. Homestake began open pit uranium mining at the Pitch Mine in 1977. Reclamation activities have been ongoing since mining ceased at the property in 1984. Slope instability in the North Pit is directly related to the response of the geologic system to pit excavation.

During the spring and again in the fall of 1983, large-scale, rapid slope failures occurred in the northeast corner of the North Pit. These appear to be plane shear translational failures that occurred in response to oversteepening of the east wall, due to the moment-driven deformation at the east wall of the North Pit, coupled with particularly high precipitation and other meteoric effects.

THE NORTH PIT

The North Pit currently extends from a maximum elevation of about 10,900 ft (3323 m) above mean sea level (amsl) at the east wall, to the level of the North Pit Lake at about 10,320 ft (3146 m) amsl. The North Pit has a maximum length of about 1200 ft (366 m) and a maximum width of about 1000 ft (305 m). It is comprised of the east wall, north wall, south wall, west wall, and the northeast corner. A shallow pit lake (North Pit Lake) currently occupies the bottom of the North Pit. Figure 1 is a photograph of the North Pit.

During mining operations, the floor of the North Pit extended to a minimum elevation of 10,220 ft (3116 m) amsl, which corresponds to a depth that is about 100 ft (30 m) below the

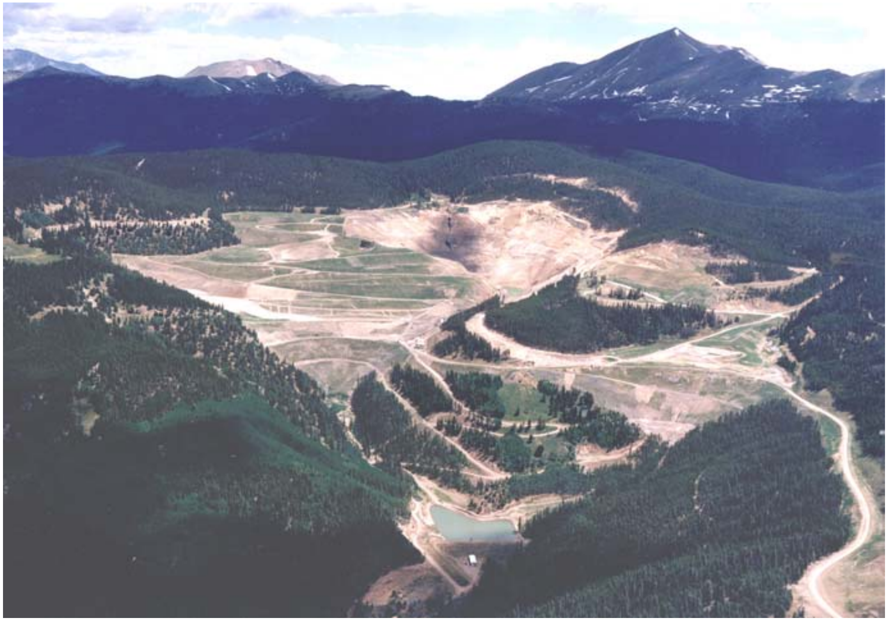


Figure 1. Photograph of North Pit looking north.

current surface of the North Pit Lake. Also during mining operations, the deepest and narrowest part of the North Pit was at the north end of the pit. The east wall and north wall were originally excavated at about 42 degrees from horizontal. The west wall was originally excavated to about 38 degrees from horizontal. The south wall was originally excavated to 30 degrees from horizontal.

As a result of slope re-grading in the North Pit in 1996, the overall slope angle of the east wall has been reduced from 42 degrees to 28 degrees from horizontal. Re-grading plans for the east wall involved first dozer-pushing the material off of the 10,600 bench, which lies at about mid-pit level, then grading the material to the level of the North Pit Lake. However, when deposition of the material below the 10,600 bench was nearly complete, the material failed and slid to the angle of repose. The failed material partially filled the North Pit Lake and translated to the opposite (west) side of the lake. The failed material resulted in significant buttressing of the east wall.

A geotechnical model of the east wall of the North Pit was developed for the purpose of slope stability analysis, as detailed in the Geotechnical Slope Model section of this paper. The lower roughly one-half of the east wall pit slope, which is comprised primarily of clay that is the result of intense sericitic alteration, is herein termed the Lower Block. The upper roughly one-half of the east wall slope, which is characterized by a series of high-angle faults that dip toward the pit, is herein termed the Middle Block. The herein termed Upper Block lies between the crest of the east wall and the prominent headscarp that lies above the east wall.

SITE GEOLOGY

The Pitch site is located on the west side of the southern flank of the Sawatch Range in the southern Rocky Mountain physiographic province. An erosional remnant of Paleozoic rocks underlies most of the site. The Paleozoic rocks extend past the western boundary of the site to a contact with volcanic rocks that are associated with Tertiary volcanism in the West Elk Mountains. The eastern boundary of the Paleozoic block is coincident with the Chester Fault Zone. The Chester Fault Zone is a roughly north–south trending, high-angle reverse fault zone of Laramide age. Igneous intrusive and metamorphic rocks extend from the Chester Fault Zone to the east flank of the Sawatch Range.

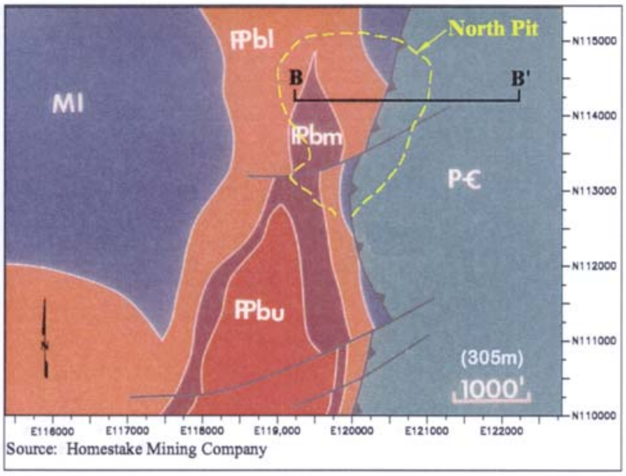
GEOLOGY OF THE NORTH PIT

Excavation of the North Pit revealed a complex geologic system. The North Pit lies along, and either side of, the Chester Fault Zone. Precambrian rocks, including pegmatite, amphibolite and schist, were thrust from east to west against a block of Paleozoic rocks, including dolomitic limestone of the Mississippian Leadville Limestone Formation (Fm), and sandstone, carbonaceous claystone, and siltstone of the Pennsylvanian Belden Fm.

The contact between the Precambrian block and the Paleozoic block is defined by the north–south trending Chester Fault. Westward thrusting of the Precambrian block against the Paleozoic block resulted in the folding, tilting, and overturning of the Paleozoic rocks. This deformation resulted in the formation of a plunging syncline in the Paleozoic block. The east limb of the syncline is overturned and was previously exposed in the north wall of the North Pit prior to pit re-grading. The west limb dips more gently to the east. The entire syncline plunges to the south at about 20 degrees. Figure 2 is a generalized geologic map and cross section of the Pitch site.

Maximum folding occurs in the North Pit area and the limbs of the syncline dip more gently to the north and south of the North Pit. The tight folding in the vicinity of the North Pit probably resulted in a greater degree of brittle fracturing in the very brittle dolomitic limestone of the Leadville Fm. The resultant increased permeability likely led to enhanced supergene mineralization and emplacement of the Pitch site pitchblende deposit in the Leadville Fm.

The Homestake exploration program revealed that the Precambrian block at the east wall of the North Pit was cut by a series of faults that strike roughly north–south and dip at high angles (60 to 70 degrees from horizontal) into the east wall of the North Pit. These faults display obsequent movement, such that the downslope side of the fault moves up, relative to the upslope side. These faulted blocks tend to restrict downgradient migration of ground water, due to low permeability fault gouge in the shear zones. In response to this dam effect, sag ponds and springs had formed on the east wall of the North Pit, with the greatest number of these features occurring at the contact between the Precambrian lithologies and the sericitic-altered block at the pit slope toe. When the east wall was re-graded in 1996, the sag ponds were drained, and the spring water was collected in a lined drainage, informally dubbed “Spring Creek,” and channeled off of the east wall.



(Figure 2A)

Explanation	
P€	Precambrian pegmatite, amphibolite, and schist
MI	Mississippi Leadville Limestone
PBB	Paleozoic Belden Formation
PPBL	Paleozoic Lower Belden Formation
PPbm	Paleozoic Middle Belden Formation
PPbu	Paleozoic Upper Belden Formation

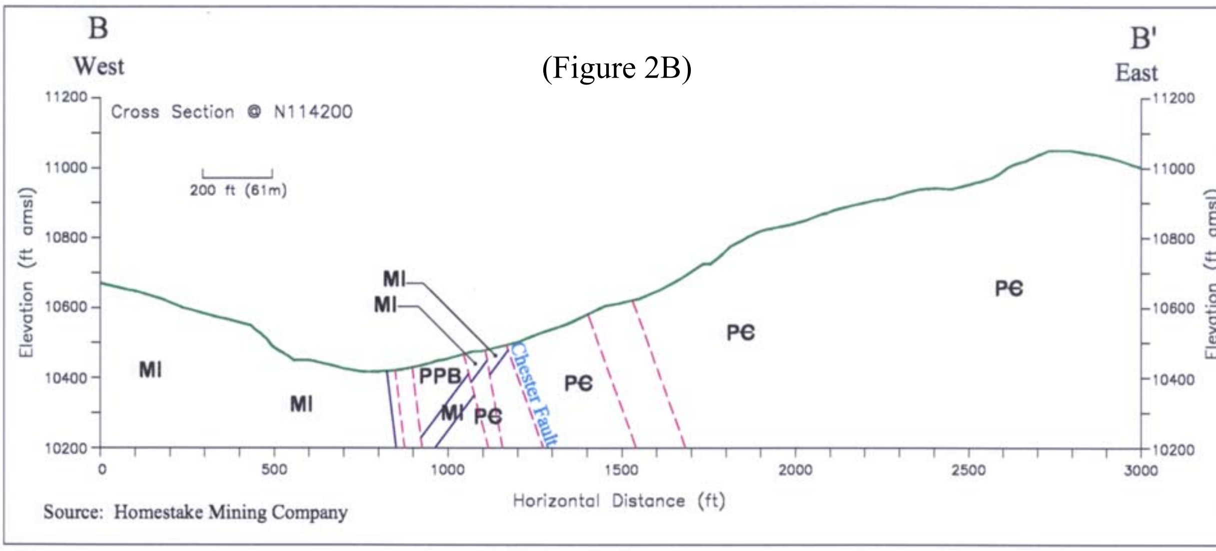


Figure 2. (A) Generalized geologic map of the North Pit area, and (B) generalized east–west, pre-mining geologic cross section through the North Pit area.

The Chester Fault Zone is also cut by a series of transverse faults that trend northeast–southwest with a strike of about 075 degrees. Vertical offset and drag folding along these faults may be observed in the east wall of the North Pit. These transverse faults may have provided a side release mechanism for the north–south fault set. Figure 3 is a geologic structure map for the Pitch site, illustrating the trend of the Chester Fault Zone and the major transverse faults.

In the slopes that lie above the crest of the North Pit, weathering has resulted in a relatively shallow zone of weak, highly weathered granite. This zone has a maximum thickness of about 150 ft (46 m). A perched ground-water zone occurs at the base of the Upper Block, which is coincident with the transition to unweathered competent rock.

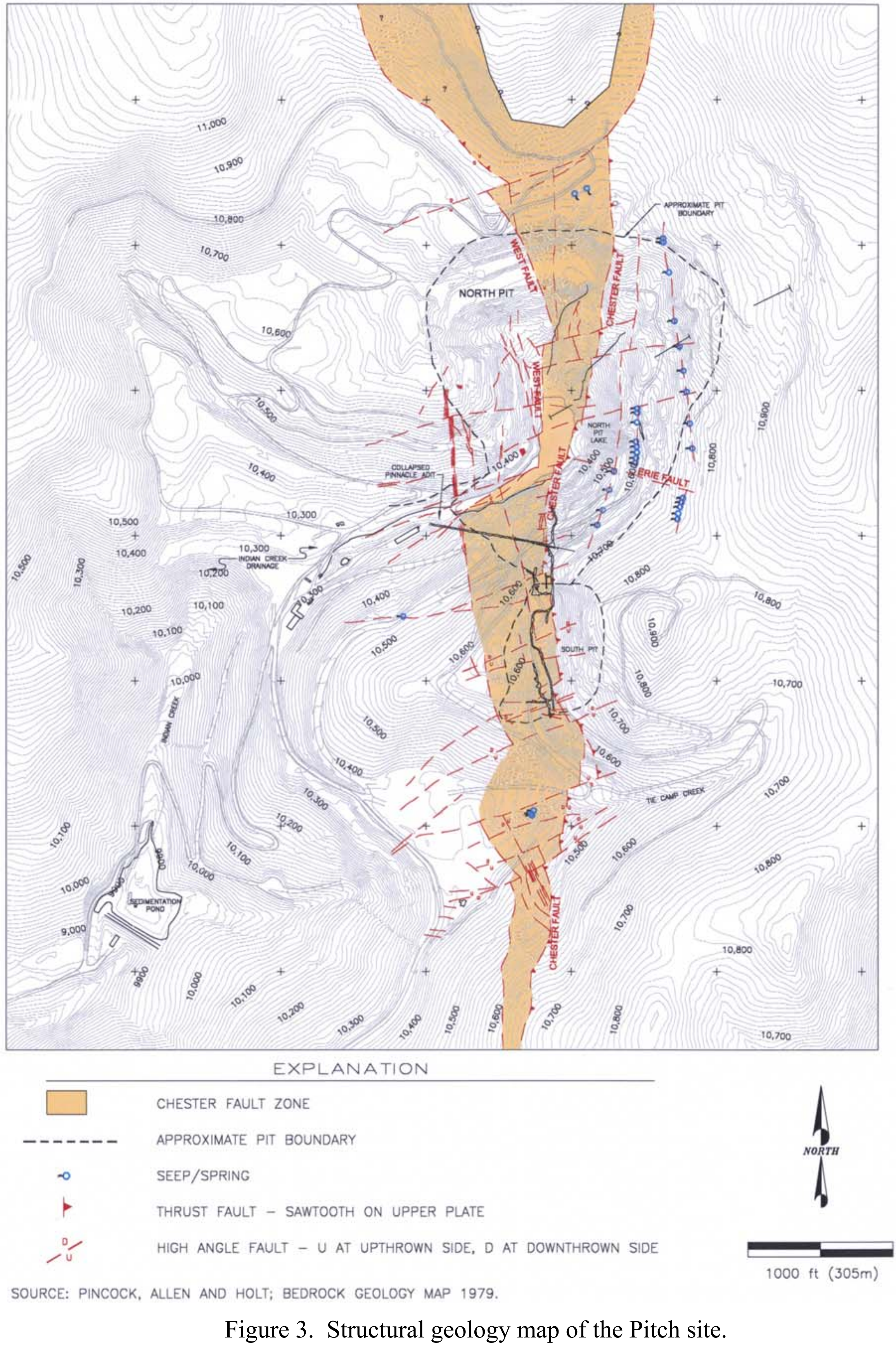


Figure 3. Structural geology map of the Pitch site.

HYDROGEOLOGY OF THE EAST WALL OF THE NORTH PIT

The Upper Block is comprised of weathered pegmatite and schist and is relatively free draining. At the Middle Block, the north–south oriented, high-angled faults create low-permeability barriers to ground-water flow. This is evidenced by north–south oriented linear patterns of springs, as shown on Figure 3. It appears that the linearity of these springs reflects the trace of high-angled, in-dipping faults. Movement along the northeast–southwest oriented transverse faults has resulted in additional barriers to ground-water flow in the north–south direction. The combination of north–south trending faults, coupled with the northeast–southwest trending faults, resulted in a compartmentalized ground-water system.

The intense alteration associated with ore deposition created a block of sericitic-altered clay at the Lower Block of the east wall of the North Pit. The sericite clay block at the toe of the east wall of the North Pit also appears to have contributed to inhibiting ground-water flow. Springs and seeps are abundant on the east wall, above the level of the contact with the sericitic clay block, but none are observed below the contact with the sericitic clay block. Piezometers installed in the sericitic clay block, which have been completed to about the level of the North Pit Lake, have been dry.

Compartmentalization of ground water in the Middle Block, due to the high-angle, in-dipping faults and the transverse faults, meant that dewatering holes (horizontal drain holes) only drained small, discrete portions of the pit wall. Dewatering of the Lower Block was ineffective because the block is comprised of low-permeability, low-strength clay. As a result, dewatering holes were often lost to collapse, and those that did drain prior to collapse yielded very little water.

HISTORY OF SLOPE INSTABILITY AT THE EAST WALL OF THE NORTH PIT

Homestake began mining operations at the Pitch Mine in 1977. By 1979, excavation of the North Pit was underway. Raveling and minor slope failures began to occur in the shallow pit walls during the first year of excavation. The first slope failure on the east wall occurred in March 1980. This was a bench-scale failure. In March 1983, a large-scale, rapid slope failure occurred in the northeast corner of the east wall. Figure 4 is an historic photograph of the March 1983 slope failure. In October 1983, a second large-scale, rapid slope failure occurred in the northeast corner of the east wall. The October 1983 failure involved about twice the volume of the March 1983 event. Figure 5 is an historic photograph of the October 1983 slope failure.

The year 1983 was a climatological anomaly, related to the El Niño weather phenomenon. The combination of warm fall temperatures (that caused the ground to remain unfrozen late into the year, enhancing infiltration), heavy winter snowpack, early and rapid snow melt and increased rainfall, resulted in optimum conditions for reducing shear strengths of slope materials in response to elevated pore water pressures. Slope displacement velocities reacted quickly to the increased infiltration of surface water and rising ground-water levels. On March 7, 1983, the slope displacement rates reached a non-recoverable velocity in the northeast corner of the North Pit and continued to increase until slope failure took place on March 13 and 14, 1983.

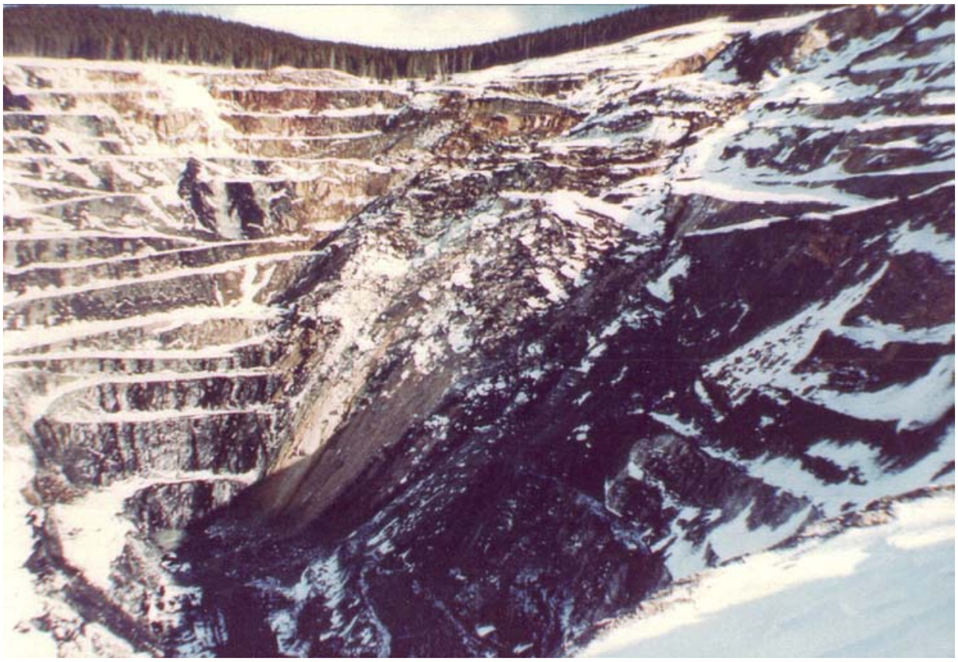


Figure 4. Northeast corner slope failure — March 1983.

Figure 5. Northeast corner slope failure — October 1983.

The east wall headscarp, which defines the eastern extents of the Upper Block, had developed as early as September 1981, and possibly earlier in pit development. The headscarp was referred to as the “tension crack” in earlier Homestake files. It is unclear from the available information whether the ground between the east wall pit crest and the headscarp (Upper Block) had developed tension cracks prior to the March 1983 event. The March 1983 slope failure in the northeast corner of the east wall resulted in about 60 ft (18 m) of vertical displacement at the headscarp. The slope failure mass appears to have originated primarily above the 10,600 bench, which traverses the North Pit at about mid-pit height. Large-scale, rapid slope failure was limited to the northeast corner of the North Pit, although headscarp development and tension cracking occurred throughout the Upper Block. The geometries of the shear surfaces of the northeast corner failures are characteristic of other slope failures in the weathered pegmatite at the site. These are characteristically circular in profile near the headscarp and roughly linear with little or no curvature at the toe.

As the North Pit slopes advanced north due to mining, following the March 1983 slope failure in the northeast corner of the North Pit, surveys of slope monitoring points revealed that all of the North Pit monitoring points were accelerating. The October 1983 northeast corner failure occurred in the same area as the March 1983 northeast corner slope failure, but progressed farther up the slope. The October event involved about twice the volume of the March event. The following account, quoted from the Homestake, October 1983, Monthly Report provides a dramatic account of the slope failure.

> "By October 13, 1983, the "swamp" area had reached over six inches a day in movement rates and failure was imminent…Friday morning, October 14, in an hour and a half, the North east (sic) corner came in…
>
> October 14, 1983: Storm front brings rain and snow to the mine during the night (0.5 inches of precip(itation))
>
> 5:00 a.m. Sump pump in the north end of the pit moved because sump filling in with mud.
>
> 7:30 a.m. Friday's crews arrive, snowing, pit shrouded in fog.
>
> 8:00 a.m. Large cracks noted in fresh snow at tree line above the swamp.
>
> 8:30 a.m. Slide well underway. Perimeter well defined by cracks in the fresh snow. Crews ordered to save equipment (pumps and light plants). Bulge noted in center of slide area.
>
> 9:00 a.m. Slide toe advancing rapidly (one ft per minute) as crews scramble to move equipment.
>
> 9:30 a.m. Crews and equipment safe. Photographs taken, slide toe stops advancing.
>
> 10:00 a.m. Crews sent home, water from the (10)600 level of slide reworking slide material as it works it(s) way into the pit. Slide stable."

The October 1983 northeast corner failure completely filled the north end of the North Pit. The North Pit Lake began forming immediately after the slope failure. Displacement at the headscarp appears, based on photographic evidence, to have roughly doubled in magnitude to a total vertical displacement of about 120 ft (37 m).

Figure 6 is a Homestake photograph of the headscarp taken after the March 1983 event. Figure 7 is a photograph from a similar perspective that was taken after the October 1983 event. Comparison of these photographs reveals the magnitude of displacement at the headscarp initiated by the two 1983 slope failures.

The North Pit Lake currently obscures the toe of the northeast corner slope failure. The failure surface above the 10,600 bench has been excavated to the apparent failure plane. Material from the north wall was pushed onto the toe of the former slide during the 1996 re-grade of the east wall.

MOMENT-DRIVEN REGRESSIVE SLOPE FAILURE

A slope that accelerates to failure is termed a progressive slope failure (Zavodni & Broadbent, 1978). From a mining or reclamation perspective, progressive failures such as the 1983 northeast corner failures are unacceptable. However, the style of slope displacement that governs the entire east wall is more accurately termed regressive failure. A regressive failure is defined as a slope that is moving toward equilibrium and is continually decelerating as the mass is readjusted or forces contributing to instability are reduced (Zavodni & Broadbent, 1978; Call et al., 1993).

Moment-driven failures were described by Nieto and Matthews (1987) as a form of deep-seated toppling. Nieto and Matthews (1987) described the kinematic geometry of a moment-driven failure in similar terms to those used herein to describe the east wall of the North Pit. Nieto and Matthews (1987) describe a “passive wedge” at the toe of a slope, which is analogous to the sericitic Lower Block of the east wall of the North Pit. The term “toppling section” is used to describe what is herein called the Middle Block. An “active wedge” is defined as the block above the toppling section, which is analogous to the Upper Block of the east wall of the North Pit.

The characteristics of moment-driven slope deformation cited by Nieto and Matthews (1987) include tension cracks and headscarp development near the crest, shear fractures with obsequent faulting in the middle portion of the slope, and a bulging toe section. These characteristics have all been observed at the east wall of the North Pit of the Pitch site. Nieto and Matthews (1987) also suggest that because the deformation involves moments, the forces involved are less than those of a translational type failure. Nieto and Matthews (1987) proposed that the forces required to establish equilibrium would also be less than those expected for a translational type of failure.

Call and others (1993) described regressive slope failure in large open pits. Although Call and others (1993) did not use the term “moment-driven failure,” the characteristics described are consistent with those described by Nieto and Matthews (1987), and have all been observed at the east wall of the North Pit. These characteristics include low-strength rock mass at the toe, in-dipping fault-bounded blocks oriented sub-parallel to the strike of the pit face with clay alteration of faults between the blocks, high-angle side release faults, and compartmentalized ground water.

Figure 6. East wall headscarp following the March 1983 northeast corner slope failure.

Figure 7. East wall headscarp following the October 1983 northeast corner slope failure.

Both Nieto and Matthews (1987) and Call and others (1993) promoted continued mining in a regressive slope environment, if slope displacement can be controlled by such means as dewatering, controlled production rate, and strategically placed stepouts. Slope monitoring is also cited as a key element to protecting personnel and equipment.

Limit equilibrium methods do not accurately represent moment-driven slope displacement because a discrete shear surface is not present. Numerical analysis is much better suited to analysis of moment-driven deformation. Cremeens and others (2000) used a two-dimensional distinct element model for the east wall of the North Pit that accounted for the rotation, bending, frictional sliding, and plastic deformation that occurred in the east wall, and allowed prediction of future slope performance.

GEOTECHNICAL SLOPE MODEL

A geotechnical model of the east wall of the North Pit was developed to facilitate numerical slope stability analyses. Details of the numerical slope stability evaluation are presented in a previous publication (Cremeens et al., 2000). The east wall of the North Pit was divided into three zones, based on rock strength and style of slope displacement. Intense, sericitic alteration associated with ore emplacement resulted in weak, plastic clay in the lower part (toe) of the pit slope, herein termed the Lower Block. The Middle Block contains a series of high-angle, in-dipping faults that strike parallel to the East Wall of the North Pit. Deformation of the Lower Block allowed rotation of the fault-bounded blocks of the Middle Block toward the pit. Weathering resulted in a contact between weak, weathered rock, and fresh, competent rock in the upper east wall of the North Pit. The ground above this contact is herein termed the Upper Block. The Upper Block appears to have displaced mostly as plane shear, but the obsequent style of faulting displayed in the Upper Block indicates that the failure was also influenced by rotation and shearing of in-dipping faults.

The low permeability sericitic clay at the toe of the slope, and low permeability fault gouge along in-dipping, north–south oriented faults, and northeast–southwest transverse faults, created a compartmentalized ground-water system that resulted in an elevated piezometric surface at the east wall of the North Pit, further exacerbating pit instability.

Figure 8 is a map of the North Pit, showing the northeast corner pit failure locations, the Upper, Middle, and Lower Blocks, and other components of the east wall of the North Pit. Figure 9 is a modeled pre-displacement profile through the most critical section of the east wall. Figure 10 is a modeled post-displacement profile showing bulging at the Lower Block, rotation of the Middle Block, and translation of the Upper Block.

CONCLUSION

The east wall of the North Pit of the Pitch Reclamation Project exhibits geologic features common to moment-driven, regressive slope deformation. These features include:

- A weak deformable toe
- A mid-pit section with high-angle, in-dipping faults oriented sub-parallel to the pit wall
- A fault set oriented transverse to the in-dipping faults

- Obsequent fault displacement
- Compartmentalized ground water

Moment-driven slope failures commonly display regressive behavior. However, the rapid, large-scale northeast corner failures of 1983 illustrate a case where a portion of a pit slope that was in a regressive mode failed suddenly in response to extreme weather-related events.

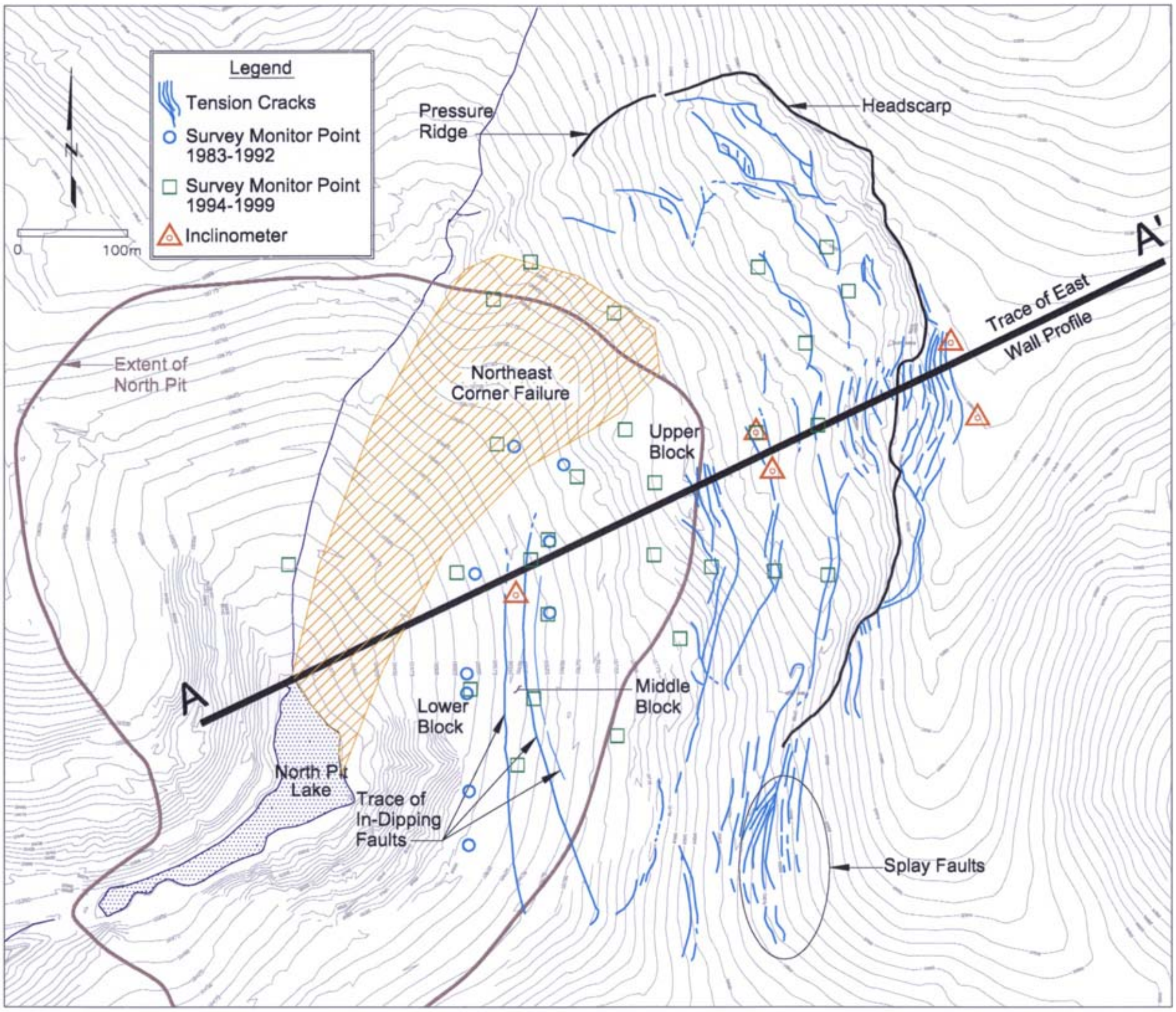


Figure 8. Map of the North Pit showing east wall components.

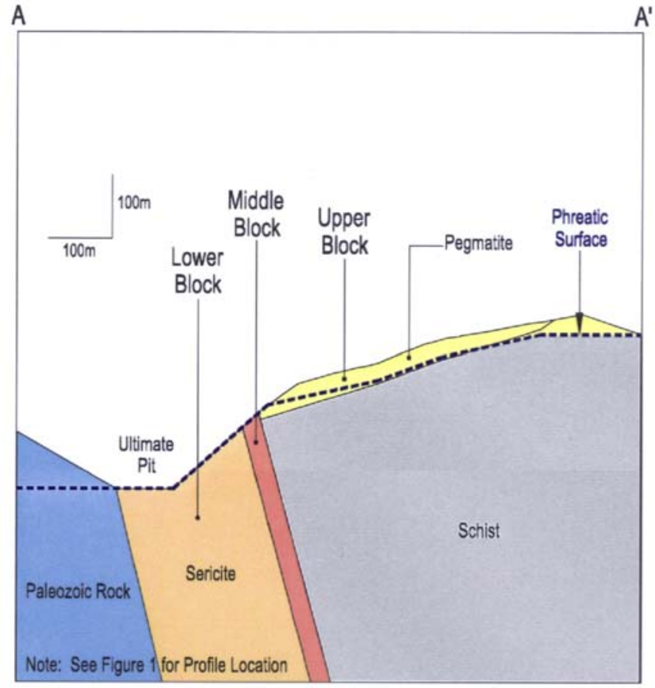


Figure 9. Pre-displacement geotechnical model profile.

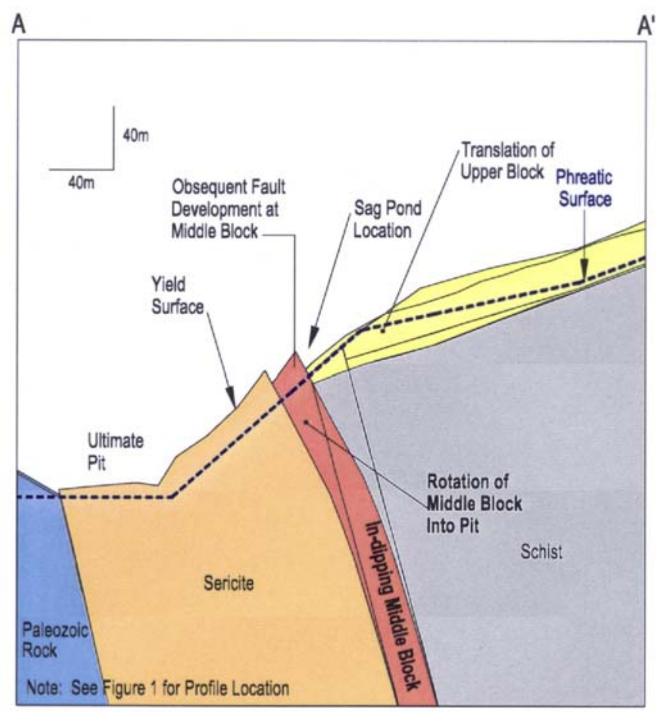


Figure 10. Post-displacement geotechnical model profile — deformation highly exaggerated.

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HISTORY OF NATURAL HAZARDS AND MINERAL RESOURCE PLANNING IN COLORADO

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Key Terms: planning, natural hazards, geologic hazards, mineral resources

ABSTRACT

This paper is intended to bring together key Colorado land use and mineral resource planning and development laws that are widely dispersed throughout the Colorado Revised Statutes. We hope to provide a detailed summary of state and local government planning regulation in order to better educate the practicing professional and improve the quality of natural hazard and mineral resource planning in Colorado. The role of the practicing professional and the Colorado Geological Survey in natural hazards planning is discussed as well as available resources.

INTRODUCTION

Colorado, like much of the country, is currently involved in a growth debate. "Smart Growth" is an important concern among local governments and citizens who experience the negative effects of unplanned growth. At times, poorly planned growth in Colorado's natural hazard areas left communities with declining property values, public safety concerns, and high costs. Communities face the need to accommodate rapid growth in difficult terrain while trying to protect community character and foster a wise use of limited natural and financial resources. Growth and the associated interest in planning are not new issues in Colorado.

Unlike some states, Colorado does not have a mandated statewide land use plan and has a long history of broad local government control. Master plans, subdivision and zoning regulations, and building codes are designated by each town, city, or county and vary in complexity and procedures. This can present quite a challenge to professionals practicing in different parts of the state.

A master plan contains a community's land use goals and visions, while zoning establishes very specific land uses. Subdivision is the process of dividing property into lots, parcels, and tracts for resale and is where most of the detailed engineering design for infrastructure is done. The building permit process is usually the last opportunity a local government has to address natural hazards and many require a site-specific geologic, geotechnical, or foundation investigation before a building permit will be issued.

The planning tools that a local community uses to address natural hazards primarily come from enabling legislation contained in the Colorado Revised Statutes (CRS). The legislation ranges from providing broad authority to outlining very specific requirements and limitations for master plans, zoning, and subdivision regulations.

LOCAL GOVERNMENT LAND USE CONTROL ENABLING ACT

The Local Government Land Use Control Enabling Act (C.R.S. § 29-20-101 through 108, from HB 74-1034) grants counties and municipalities broad authority to plan for and regulate the use of land, with no restrictions, conditions, or procedures prescribed for local governments. Each local government has the authority to plan for and regulate the use of land by:

- Regulating development and activities in hazardous areas;
- Regulating land use on the basis of the impact thereof on the community or surrounding areas;
- Otherwise planning for and regulating the use of land to provide planned and orderly use of land and protection of the environment. (Department of Local Affairs, 2001)

There are limitations to the broad local government authority due to constitutional private property rights and compliance with other statutory requirements. The statute is limited in applicability due to its broad authority; lack of standards, criteria, and specific guidance. (Johnson and Himmelreich, 1998)

COLORADO LAND USE ACT

One point discussed in the recent growth debate is that Colorado has not passed any comprehensive land use legislation in the last 30 years. In 1970, the first major land use efforts resulted in the Colorado Land Use Act (C.R.S. § 24-65-101 through 106). The act states, "The rapid growth and development of the state and the resulting demands on its land resources makes new and innovative measures necessary to encourage planned and orderly land use development."

The act established the Colorado Land Use Commission (LUC) as the primary state land use planning agency. The Governor appoints the members of the commission. The LUC has not had funding and staff for over 20 years and does not truly function as a planning agency for the state. One of the duties of the LUC was to create model regulations and implementation strategies that could be used statewide but leave land use decision making to local governments. (Johnson and Himmelreich, 1998)

1041 Powers

The Colorado Land Use Act was amended in 1974 by the passage of House Bill 1041 (C.R.S. § 24-65.1-101 through 204). HB 1041 is a comprehensive planning tool that encourages local governments to designate areas and activities of state interest and to regulate development within those areas through a special permitting process. Some of the areas and activities are as follows:

Areas
- mineral resource areas;
- geologic hazard areas;
- flood hazard areas;
- wildfire hazard areas;

Activities
- site selection and construction of major new domestic water and sewage treatment facilities and major extension of existing facilities;
- site selection and construction of solid waste disposal sites;
- site selection of airports, major highways, new communities, utilities, and some water projects.

State agencies are given the primary responsibility of providing technical and financial assistance to local governments in implementing HB 1041. The Colorado Geological Survey (CGS) provides technical assistance on the identification of geologic hazards, the review of geologic reports, and the identification of mineral resource areas.

One of the important aspects of the bill is that it initially provided funding to local governments to map hazards and create hazard plans and regulations. Many counties and municipalities used HB 1041 funds to map geologic hazards and to adopt geologic hazard ordinances, plans, and regulations. CGS maintains a list of "1041" maps and has copies of many of the maps. Some cities and counties also have copies of the maps that are available to the public.

Natural hazards, including geologic hazards, are defined in HB 1041 (C.R.S. § 24-65.104), and CGS was charged with creating model geologic hazard area control regulations and guidelines for geologic reports. CGS Special Publication 6 (Rogers and others, 1974) contains the model geologic hazard regulation. CGS Special Publication 12 (Shelton and Prouty, 1979) lists engineering geology report guidelines.

HB 1041 can be an effective hazards planning tool when used by local governments. However, local governments are not required to implement 1041 regulations and many feel that the detailed statutory requirements are too cumbersome or take away local land use control. In addition, financial assistance to local governments for implementation of HB 1041 has not been provided since the 1970s, and its use as a planning tool has been limited.

Notification of Surface Development

Mineral and surface estates are sometimes severed, and conflicts can occur between the two interests. In 2002, the Colorado Land Use Act was again amended to establish a procedure for providing notice of potential surface development to owners of mineral interests. Developers submitting an application for a preliminary or final plat for a subdivision or planned unit development must provide 30 days notice, of the first public hearing, to mineral estate owners.

The article establishes procedures for notification to mineral estates owners. The surface owner has the responsibility to research county records for mineral estate owners. If the surface owner follows the procedures established in the article, approval of the development application cannot be restricted, curtailed, or rescinded because of failure to provide notice to mineral estate owners.

If the notification procedures are not followed, the mineral estate owner can seek compensatory monetary damages. Mineral estate owners must commence court action within one year after final approval of the development by the local government or within 60 days after the start of construction, whichever is later.

MASTER PLANS

Master planning is the creation of policies and recommendations that guide land development within a community. Sometimes the terms “master plan” and “comprehensive plan” are used interchangeability but they are different types of land use planning. In Colorado, comprehensive planning generally includes a delivery of services element and social factors, such as affordable housing, and most local governments have the authority to prepare master and comprehensive plans. Generally, master and comprehensive plans are only advisory documents and are not enforceable (Elliot and Mugler, 1992). However, local governments have the authority to make master plans enforceable by including plan elements in their land use regulations and some have chosen to do so (Hill, 2003). The following discussion of the statutory basis for natural hazards planning will focus on master planning.

Municipal Master Plans

Cities and towns may create planning commissions, sometimes called planning boards in home rule municipalities, that make and adopt master plans (C.R.S. § 31-23-206 through 209). Planning commissions must consider the following elements when creating a master plan; however, they are not required to include them in the plan:

- A master plan for the extraction of commercial mineral deposits;
- Areas containing steep slopes, geological hazards, wetlands, floodways and flood risk zones, highly erodible land or unstable soils, and wildfire hazards.

To determine the location of such areas, planning commissions are directed to consider the following sources for guidance:

- The Colorado Geological Survey for defining and mapping geological hazards;
- The Natural Resources Conservation Service for defining and mapping unstable soils and highly erodible land;
- The Federal Emergency Management Agency (FEMA) for defining and mapping floodplains, floodways, and flood risk zones.

During the 2001 legislative session, sections (C.R.S. § 31-23-206 (4) and (5)) were added to the statute to make master plans mandatory for municipalities with a population of two thousand or more. However, the mandatory master plan must only address the recreational and tourism needs of residents.

County Master Plans

In terms of natural hazards planning, the requirements for a county master plan are similar to those for municipalities (C.R.S. § 30-28-106). Counties must consider natural hazards but are not required to include hazards in their plans. As with municipalities, most counties are now required to adopt a master plan; however, the plan must only contain tourism and recreational elements.

Master Plan for the Extraction of Commercial Mineral Deposits

In 1973, due to concerns about a shortage of sand, gravel and quarry aggregate, the general assembly passed legislation (C.R.S. § 34-1-301) that requires counties, with a population of least 65,000, to conduct a study of commercial mineral deposits in their county and develop a master plan for the extraction of such deposits. The intent of statute was to allow for the extraction of commercial mineral deposits while protecting the environment and citizens.

The Colorado Geological Survey was required to identify and locate sand, gravel, and quarry aggregates resources in populous counties. Special Publication 5B, which is titled "Sand Gravel and Quarry Aggregate Resources, Colorado Front Range Counties", contains the results of the resources study (Colorado Geological Survey, 1974). CGS has continued to identify potential oil and gas, coal, metallic, industrial, and construction industry mineral resources in publications and in studies for counties and the Colorado State Land Board.

Counties are prohibited from permitting land uses that would interfere with the extraction of a commercial mineral deposit (C.R.S. § 34-1-305). A "commercial mineral deposit" is defined as a "commercially feasible" and "economically significant" coal, sand, gravel or quarry aggregate deposit. This definition is somewhat objective and the value of a developed subdivision can exceed the value of an extracted mineral deposit. In such cases, counties often consider a mineral deposit commercially infeasible and insignificant and allow land uses that interfere with the removal of the deposit.

During zoning, many counties require that the developer demonstrate that the site does not contain a commercial mineral deposit. Depending upon the county and the nature of the deposit, this can vary from a simple statement by a geologist, geotechnical engineer, or real estate agent to a detailed resource inventory and appraisal by a certified appraiser.

ZONING

Zoning is the classification of an area into zones or districts to regulate land and the construction of buildings and other improvements. Zoning is the traditional tool local governments use to balance public interests with private property rights. Its general purpose is to allow specific land uses and buildings only in specific areas. The authority to zone is an example of a local government's police powers or the power to regulate activities to protect the health, safety, and welfare of the public.

Traditionally, zoning has been used to control building heights, setbacks, and lot coverage. However, overlay zoning has been used to control development in areas with wetlands, steep slopes, and natural hazards. An overlay zone establishes special procedures and requirements that must be met before uses in the underlying zone district are allowed.

For example, in order to obtain a building permit in a platted residential subdivision, one would normally only have to meet the zoning requirements of that particular residential zoning. However, if an overlay zone were in place, the requirements of the overlay zone would need to be met in addition to the requirements of the underlying zoning. Many municipalities and counties in Colorado regulate development, in areas with expansive bedrock, geologic hazards, and steep slopes, with overlay zoning.

COUNTY SUBDIVISION REGULATIONS

Since 1972, counties have been required (C.R.S. § 30-28-133 through 133.5, from SB 35, 1972) to develop subdivision regulations that apply to divisions of land in unincorporated areas that result in parcels of less than 35 acres. Every county is required to create a planning commission, although in smaller counties the board of county commissioners may act as the planning commission. Planning commissions develop the subdivision regulations that are adopted by county commissioners (Department of Local Affairs, 2001). County subdivision regulations must include natural hazards mitigation and are the primary tool most counties use to control development of hazardous areas already zoned.

Subdivisions platted before 1972 are excluded from having to comply with county subdivision regulations. These subdivisions, many of which have steep mountainous terrain and numerous natural hazards, continue to be a problem for local governments and are a heavily debated issue for lawmakers. However, a few communities have used overlay zone districts and/or HB 1041 to address development of older subdivisions.

Some of the plans, reports, designs, and studies required with a subdivision plan are listed below:

- Reports that detail the geologic characteristics of the area and how those characteristics affect the proposed land use;
- Evaluation of potential radiation hazards;
- Maps and reports that address the suitability of soil types for the proposed land use;
- Plans for sewage disposal where no central sewage treatment facility is proposed and the suitability of the sewage disposal method;
- Plans for storm water facilities to prevent storm waters in excess of historic runoff;

County commissioners cannot approve a preliminary or final plan or plat unless the required reports, plans, and studies have been submitted and meet the requirements of the subdivision regulation.

The statute further states that county commissioners cannot approve a subdivision unless the developer has submitted the following:

- Evidence that sewage disposal systems comply with state and local laws and regulations;
- Evidence to show that hazards and constraints due to soil, drainage, and geologic conditions, have been identified and that the proposed land uses are compatible with the identified hazards and constraints.

In other words, a developer must identify hazards and constraints, submit mitigation plans, or show the proposed uses will not be affected by the hazard or constraint. Designating a geologic hazard area as passive open space would be an example of a compatible land use.

Counties must send a copy of a preliminary plan or plat to various local and state governments for review and comment (C.R.S. § 30-28-136). A partial list of the required referral agencies includes:

- The local conservation district for review and recommendations regarding soil suitability, flooding, and watershed protection;
- The Colorado Geological Survey for an evaluation of geologic factors that have an impact on the proposed land use;
- The county or regional health departments for review of on-lot sewage disposal and the water quality of proposed water supplies reports.

The local health department may require additional engineering or geological reports that are not required by subdivision regulations. Some counties do not have a local or regional health department and in those counties, the planning department normally reviews domestic sewage disposal plans. County commissioners cannot approve a preliminary plan unless the local health department has approved the proposed method of sewage disposal.

Rural Cluster Process

Recently, the general assembly established the rural cluster process that allows single-family residential parcels, less than 35 acres, without a full subdivision process. This allows a developer or landowner to earn double the density allowed under Senate Bill 35, or up to one dwelling unit per 17.5 acres, if two thirds of the land is reserved for open space.

The intent of the rural cluster process is to streamline the subdivision process for landowners who cluster development and preserve open space. Depending upon the county, a developer is required to comply with part or all of the county's subdivision regulations. The requirement to address natural hazards varies among counties with a rural cluster process.

MUNICIPAL SUBDIVISION REGULATIONS

Cities and towns are not required to adopt subdivision regulations or include natural hazards mitigation in their subdivision regulations. Cities and towns are also not required to submit subdivision applications to agencies such as the Colorado Geological Survey and the local conservation board. However, many municipalities have chosen to address potential hazards in their land use plans and regulations and refer subdivision applications to state and local agencies for review of potential natural hazards.

The Municipal Major Activity Notice (C.R.S. § 31-23-225) requires that the county, Land Use Commission, and State Geologist be notified of any municipal activity, which covers five acres or more, before approving any associated zoning change, subdivision, or building permit application (Department of Local Affairs, 2001).

BUILDING CODES

Counties (C.R.S. § 30-28-201) and municipalities (C.R.S. § 31-15-601) may adopt building codes in order to protect the public health, safety, morals and general welfare. County and municipal building codes do not apply to school construction. Schools must meet building standards set forth by the Colorado Department of Labor and Employment (C.R.S. § 22-32-124). If a county or municipality does not have a building code, certain buildings intended for multiple occupancy, such as motels, hotels, and schools, are subject to building standards set forth by the state Division of Housing (C.R.S. § 24-32-709, et seq.) (Department of Local Affairs, 2001).

Some local governments require a site-specific soils and foundation investigation before a building permit will be issued and have standards that the investigation must meet. In addition, a grading plan and/or permit, slope stability analysis, and other studies are often required in areas with natural hazards or steep slopes. For example, Jefferson County has special foundation design criteria in their building code for areas located in the Designated Dipping Bedrock Area (DDBA). The DDBA is an overlay zone district that regulates development and building codes in an area with highly expansive and steeply dipping claystone bedrock.

Many times, the special studies and reports that need to be submitted with a building permit are not included in the building code but are a result of a special condition or note included on a subdivision plat. When performing a soils and foundation investigation, it is prudent for the practicing professional to review the plat for any special conditions or requirements.

For example, a subdivision, on Green Mountain in Jefferson County, was proposed in an area well known for its instability. The subdivision plat contained a note requiring a slope stability analysis for each lot and other special conditions such as xeriscape landscaping. Homes, large retaining walls, fill, and improper landscaping were constructed without the required slope stability analyses. Subsequently, an older landslide reactivated, in part due to improper construction (Thompson, 1998), and five homes were destroyed or damaged. Many of the geotechnical engineers who performed investigations for foundations or designed retaining walls were unaware of the plat note requiring slope stability analyses or that the lots was located on a mapped landslide.

Soil Hazard Disclosure

Growth is occurring in geologically sensitive rural areas as well as within urban areas. Development is occurring in areas that were historically avoided due to potential hazards. Damage from natural hazards, such as expansive soils and bedrock, is steadily increasing in Colorado due to development pressure.

In response to widespread damage to homes from expansive soils, Colorado passed a disclosure law under the consumer protection act. Titled “Soil and Hazard Analyses of Residential Construction” (C.R.S. § 6-6.5-101), builders must provide purchasers with a copy of summary geotechnical report and site recommendations fourteen days before closing. Failure to provide the required information can result in a $500.00 penalty. The statute is sometimes more broadly interpreted to include other geologic hazards (Johnson and Himmelreich, 1998).

PUBLIC SCHOOL SITES AND BUILDINGS

Colorado public school districts have the authority to determine the location of public schools and to construct schools and other structures (C.R.S. § 22-32-124). Before purchasing land or constructing buildings, school districts are required to consult with CGS regarding swelling soils, mine subsidence, and other geologic hazards.

In addition, school districts must consult with the local planning commission before contracting for the purchase of land or building structures. Often, the local planning commission will request geologic and engineering reports and designs for drainage, grading, erosion and sediment control, etc., and will review a school district’s proposal for conformance with comprehensive plans. School districts do not have to follow the recommendations of CGS or the local planning commission but most implement recommendations for natural hazards mitigation.

RESPONSIBLITIES OF PRACTITIONERS

Qualifications

Colorado does not have registration or licensing requirement for geologists. However, there are minimum statutory requirements for practicing professionals (C.R.S. § 34-1-201 through 202, from HB 1574, 1963). Any legally required report prepared as a result of or based on a geologic study or on geologic data, or which contains information relating to geology and which presented to or prepared for any state agency or political subdivision of the state, such as a local government, shall be prepared or approved by a professional geologist.

The statute defines a "professional geologist" as: a person who is a graduate of an institution of higher education which is accredited, with a minimum of thirty semester (forty-five quarter) hours of undergraduate or graduate work in a field of geology and whose post-baccalaureate training has been in the field of geology with a specific record of an additional five years of geological experience, to include no more than two years of graduate work (C.R.S. § 34-1-201).

Professional Standards of Practice

In Colorado, there is an issue of interest to geologists and engineers that concerns the boundaries and overlap between the two practices. The limits of professional practice for geology and engineering are of interest because engineering geologists and geotechnical engineers often perform overlapping functions. Some practitioners qualify as both, but most are either a geologist or an engineer by training. There is concern that reports containing geologic information are not being prepared or approved by a qualified geologist, as defined in HB 1574. Conversely, there is concern that geologists and not engineers are reviewing engineering reports.

Several notable activities have occurred recently in Colorado in response to this issue:

- The Colorado Board of Registration for Professional Engineers and Professional Land Surveyors (CBRPEPLS) has published Policy Statement 50.2, which lists key guidelines and limitations for engineering in designated natural hazard areas. A task force of engineers and geologists updated the policy statement in 2000.

- CBRPEPLS solicited the input of various professional engineering and geological organizations and agencies, in 1998, regarding the adoption of documents from the California Board of Registration for Professional Engineers and Land Surveyors. The California documents suggest job activities done by professional engineers, those done by professional geologists, and those shared by both professions. Because of the controversy generated by those documents, CBRPEPLS asks that engineers and geologists engage in a dialog concerning the limits of professional practice, and is willing to facilitate these discussions.

ROLE OF THE COLORADO GEOLOGICAL SURVEY IN NATURAL HAZARDS PLANNING

Land Use Reviews

The Colorado Geological Survey evaluates geologic factors that would have significant impact on the proposed use of the land for subdivision purposes by reviewing preliminary plat or plan applications. The agency conducts a variety of special-use reviews and provides technical assistance to county and city governments, school districts, water and sanitation districts, and other government agencies upon request. Subdivision reviews account for a majority of CGS review activities. House Bill 1572 (1983) mandates that CGS establish and collect fees to recover the direct costs of providing review services.

For most cases, the CGS receives and reviews geologic-suitability reports (under various titles such as “geologic” or “geotechnical” reports), drainage reports, and plat maps submitted for proposed subdivisions. A CGS engineering geologist visits the actual subdivision site and performs a reconnaissance in order to check the submitted information. The reviewer then writes a review letter to the local government planning agency from which the submittal packet was sent. There are four basic levels of response:

- The submitted findings and recommendations are completely adequate;
- The submitted findings and recommendations are mostly adequate, and additional suggestions are given;
- More information is needed because potentially serious geologic problems were not sufficiently recognized or addressed; or
- The project is infeasible for geologic and/or other or technical reasons.

CGS reviews are advisory in nature, and are therefore non-binding. The local planning agency may choose to disregard the CGS review, although this is seldom the case. The extent of the review is determined primarily by the stage of planning, complexity of the project, and/or the severity of geologic constraints. Each site will have unique geologic conditions, and must therefore be investigated and reported accordingly.

For a preliminary plat report, the geologic investigation should go beyond a simple reconnaissance; it should be a solid, preliminary level investigation that addresses subsurface as well as surface conditions. Please refer to CGS Special Publication 12 (Shelton and Prouty, 1979) and the local government's subdivision regulations for what should be included in a preliminary plat report.

Publications

CGS has published numerous books, reports, and maps that may be used in conjunction with land use planning. The information contained within these publications ranges from general to site-specific in scope, and may address single or numerous topics. CGS publications are available through the CGS publication office, at (303) 866-3340, or on the CGS web page (http://geosurvey.state.co.us). Some of the most useful geologic hazard planning publications are listed below:

- *Special Publication 6, Guidelines and criteria for identification and land-use controls of geologic hazard and mineral resource areas*
    - The publication contains the model guidelines, created under HB 1041, for use by local governments in their land use regulations. The book lists qualifications for professional geologists, engineering geologists, and professional engineers, as well as the responsibilities of geologists and engineers with respect to technical-report preparation (Rogers and others, 1974).
- *Special Publication 43, A guide to swelling soils for Colorado homebuyers and home-owners*
    - This book is geared toward satisfying the disclosure requirements for new-home buyers in accordance with SB 13. The book substantially updates and replaces two older CGS publications, Special Publications 11 and 14 (Noe and others, 1997).

- *Information Series 47, Geologic Hazards Avoidance or Mitigation*
    - This booklet contains a compilation of pertinent land-use and professional-practice laws in Colorado that deals with geology and geologic hazards. It contains excerpts from other CGS publications (Johnson and Himmelreich, 1998).
- *Solving land-use problems*
    - This free booklet is geared toward planners and developers. It describes geologic hazards and subdivision reviews conducted by CGS, and is periodically updated to provide the latest information to the public (Soule, 2003).
- *CGS Mine Subsidence Library*
    - On behalf of several federal and state mining agencies, the CGS maintains a library of coal mine and associated ground subsidence hazard reports and maps for use by geologic and engineering consultants. Copies of these materials are available to researchers upon request, at a minimal cost.
- *Information Series 60A, Colorado Late Cenozoic Fault and Fold Database and Internet Map Server*
    - An interactive map that displays color-coded faults within Colorado that is available at the following web address: http://geosurvey.state.co.us/pubs/ceno/index.htm (Widmann and others, 2003).

SUMMARY

Natural hazards, including geologic hazards, are an important consideration for land use and development activities in Colorado. This paper outlines many of the pertinent statutes for geologic suitability assessment. It also discusses the role of the Colorado Geological Survey in providing technical assistance to local government agencies and technical information to the private sector and the public.

Local governments have broad authority to regulate land uses in natural hazard areas, and each local government has developed its own procedures and regulations. In order to provide quality service to a client and protect the public, a practicing professional should know the requirements of the local government, what is required in a geologic suitability assessment and the qualifications required of a practicing professional.

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BREAKING NEW GROUND IN MOUNTAIN COMMUNITIES: LAND PLANNING AND DEVELOPMENT IN HAZARDOUS TERRAIN

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ABSTRACT

Mountain communities, by definition, are typically located in areas of rugged natural terrain. Developers working in areas of geologic hazards face a number of hurdles, but these can be surmounted with foresight and planning. Public perception of the hazards, which can impact marketing of property, reluctance of public agencies to approve development in known hazardous areas, technical design issues, and risk of liability for damage are all factors that must be considered in early planning stages of the project. This paper presents an updated approach to the planning and engineering process for development in geologic hazard areas. Case studies are presented to illustrate the planning and development process for hazardous areas.

INTRODUCTION

Mountain properties, by definition, are typically located in areas of rugged natural terrain. It is the rugged terrain that gives these areas their natural beauty and makes them a desirable resort environment. However, mountain slopes and valleys often have inherent hazards to development. In some cases hazards such as rockfall, debris flows, flash flooding, and avalanches can be of obvious and immediate danger to inhabitants. Other more subtle hazards can also result in severe damage to property and structures. Slower-moving landslides may not be visible to the casual observer, but can tear a building apart over time, resulting in severe property damage, loss of land value, and litigation.

In the past, when land was plentiful in resort areas, potentially hazardous locations could be avoided at little cost. However, with escalating land values and increasing density in resort areas, the inefficient use of land by wholesale "avoidance" of potential hazard areas is no longer acceptable. Furthermore, landslide terrain is often desirable, as natural processes have created rolling terrain, natural ponds, and aspen groves. In some areas, landslide terrain, debris deposits, and alluvial fans may afford the most advantageous, or the only sites to avoid unacceptably steep slopes.

Developers working in areas of geologic hazards face a number of hurdles, but these can be surmounted with foresight and planning. Public perception of the hazards, which can impact marketing of property, resistance of public agencies to allow development, technical design issues, and risk of liability for damage are all factors that must be considered in early planning stages of the project. An engineering geologist/geotechnical engineer (a dual-qualified individual or two specialists working in collaboration) can be of immense value to the developer/planner team, providing guidance for site use strategies, planning mitigation, and facilitating the permitting process with the local government. The greatest benefits are provided by an engineering geologist/geotechnical engineering specialist who has proven expertise in solving hazard problems, not just identifying the problem.

A REASON FOR EVOLVING THE PRACTICE

Traditional approaches to development in geologic hazard areas have been biased toward avoidance. This is largely because geologic hazard conditions are often difficult and/or costly to define and mitigate. Traditionally, additional conservatism in assumptions and design may be used to compensate for the lack of a more detailed understanding. Marginal sites are generally avoided. Additional investment in site characterization and study is needed to close the gap of uncertainty, and may allow responsible development of sites which initially may appear marginal. With decreasing available land and increasing land values, the cost/benefit climate is compelling the practice to evolve, with justifiable investment in a better understanding of potentially developable sites.

CURRENT REGULATIONS PERTINENT TO GEOLOGIC HAZARDS

To be successful in permitting a project in a known hazard area, the developer/planner must work in cooperation with the local government agency responsible for the planning review process. The planning process should include a thorough review of applicable regulations that may apply to special site conditions, including geologic hazards. Geologic hazard mitigation is mandated by state legislation, but is enforced through county master planning, zoning and subdivision processes. County review staff will often enlist the assistance of state resources for permit review. It is therefore highly beneficial for a project development team to include a specialty engineering geologist/geotechnical engineer having good rapport with the state geologic agency.

In Colorado, the Colorado Geological Survey (CGS), part of the Colorado Department of Natural Resources, is the lead agency for handling geologic hazard issues. An engineering geology group within the CGS was developed to address the serious geologic problems associated with rapid development in mountainous regions of the state, which began in the late 1960s. Several pieces of legislation outline the specific functions of the CGS, and also describe the role of Counties.

House Bill 1041 (C.R.S. §24-65-101 thorugh 106, 1974), the "Colorado Land Use Act" (C.R.S. §24-65.1-101 through 204, 1970)

This legislation charges local governments with legal responsibility for designation and administration of geologically hazardous areas of state interest, and charges CGS with preparing and publishing a set of guidelines and model regulations for local governments. In response to this charge, CGS issued Special Publication 6, *Guidelines and Criteria for Identification and Land-Use Controls of Geologic Hazard and Mineral Resource Areas* (Rogers, et al.,1974) to provide such guidelines. Several other CGS publications, which address these same issues, include:

- *Colorado Landslide Hazard Mitigation Plan*, Bulletin 48, (Jochim, et al., 1988)
- *Landslide Loss Reduction: A Guide for State and Local Government Planning*, Special Publication 33, (Wold and Jochim, 1989)
- *Geologic Hazards Avoidance or Mitigation*, Information Series 47, (Johnson and Himmelreich, 1998)

Senate Bill 35, the "Subdivision Law" (C.R.S. § 30-28-133 through 133.5, 1972)

This law requires that subdivision proposals be evaluated for geologic conditions prior to county approval, and applies to tracts or parcels of less than 35 acres. Counties are required to request site data, and sound planning and engineering requirements must be met prior to approval. No preliminary or final plats may be approved until hazardous conditions requiring special precautions are identified, and the proposed site uses are determined to be compatible with site conditions.

For development of "use by right" parcels of 35 acres or more, these requirements do not apply. However, in areas where land values are high, sale of 35-acre parcels is often not practical.

House Bill 1034, the "Local Government Land Use Control Enabling Act" (C.R.S. § 29-20-101 through 108, 1974)

This legislation gives authority to local governments to plan and regulate the use of land within their jurisdictions, including regulation of development and activities in hazardous areas.

Senate Bill 13 (C.R.S. § 6-6.5-101, 1984)

Requires residential developers to analyze and disclose any potentially hazardous conditions to prospective home buyers. Developers must also protect themselves from liability that could arise from hazardous conditions, which threaten public safety or result in property damage.

House Bill 1574 (C.R.S. § 34-1-201, -202, 1973)

Defines qualifications of Professional Geologist, and requires that all geologic reports be prepared by one. Selection of a Professional Geologist to prepare the required reports is left to the discretion of the agency or private entity contracting the work.

House Bill 1045 (C.R.S. § 22-32-124, 1984)

Requires that the board of education consult with CGS regarding geologic suitability of land prior to acquisition or construction of school buildings.

THE PLANNING REVIEW PROCESS

The regulatory review process for development in potentially hazardous areas follows the same format as for any development of its type. The process often begins with presentation of a "sketch plan," showing a general plan of the layout and extent of the proposed facilities. If the area has been previously identified as potentially having geologic hazards, it should be recognized at this initial planning stage. A diligent planner will have obtained existing information concerning such conditions, and will get assistance from an engineering geologist to identify the implications for the planned development. Where agency review personnel have prior knowledge of the site, they too will be interested in the proposed strategy to address the geologic hazard concerns. At this level of planning, generally a site field reconnaissance is completed by the engineering geologist along with a review of existing site information. The engineering geologist/geotechnical specialist makes recommendations to the planner for suitable approaches to geologic hazards mitigation/avoidance compatible with the intended site use.

Later phases of planning submittals must include information, at a level of detail corresponding to the stage of the project in the review process, and on how the geologic hazard conditions are to be mitigated or avoided. The proposed measures are generally given review by a CGS specialist, and often the geotechnical consultant will work directly with CGS to assure that the proposed mitigation plan will be acceptable.. During preliminary design, a general site investigation will be performed to further understand the geologic hazard conditions, corroborate the findings of the site reconnaissance, and provide some general geotechnical information pertinent to foundation conditions. Often, monitoring instruments are installed during the site investigation, and continued monitoring for some period of time may be incorporated as a requirement of the permit.

Detailed design of mitigation measures is generally deferred until the final design of project infrastructure required for the final plat, but the mitigation concept is generally approved by review agencies as part of the preliminary plat approval process. Often, explicit requirements for the mitigation measures are incorporated into the final plat documents in the form of a note on the plat.

REVIEW OF TRADITIONAL APPROACHES

Past practices for addressing geologic hazards and related risks to development in hazardous terrain have typically followed an approach that includes assessment and avoidance of potentially hazardous areas. With this approach, the presence or possible presence of hazards is assessed, whether by professional evaluation or "common sense," and the affected property is left undeveloped.

This approach has served the industry quite effectively in the past, for several reasons:

1. Until recently, fairly large tracts of undeveloped land have been available around expanding mountain resort and residence areas. Even if some tracts are "eliminated" by the traditional approach to hazard assessment, there have been plenty of alternate sites or tracts available for consideration.
2. Even if no alternate sites were available, the costs of mitigation could not be borne by the potential property values.

However, undeveloped (but developable) raw real estate around many expanding mountain communities is becoming scarce. For the same reason, land prices and real estate values (whether for raw land or developed lots) have risen substantially, especially during the 1990's. Affluence during the 1990's has also driven up demand for certain properties that are in short supply in many mountain communities. The result of these trends is a rather classic "supply and demand" situation. The supply is low, demand is high, and prices go up. The traditional approach is no longer the best approach in terms of optimizing land use.

A MORE EFFECTIVE APPROACH

Since the late 1990's, the authors have worked on several properties in Eagle County. As this work progressed several facts became apparent:

1. There was general agreement among previous consultants that geologic hazards, primarily in the form of ancient landslide complexes, were present on these properties. However, initial movement of most of these landslides is thought to have developed during warming, melting, and wet climate conditions following the last period of glaciation in Colorado, generally taken as 10,000 to 12,000 years ago. It is important to distinguish between ancient landslide deposits which are currently stable, and those which have experienced recent movements.
2. Other than the hazard situation, these properties were highly desirable. Location, views, and accessibility contributed to developed, saleable values in excess of $1 million per ½ acre residential lot.
3. Although there was general agreement on the geologic history of the properties, there was less consensus on existing and future landslide risks.
4. Property values had risen to the point that a substantial and costly mitigative effort could be considered.
5. Avoidance of potentially hazardous terrain was resulting in land use that was no longer efficient, loss of potential revenue, and driving "sprawl" further down the valley.

As a result of this understanding, the authors initiated strategies to address landslide issues for development of several properties. The elements of these strategies consisted of a step-wise process, in coordination with planning, permitting, and development. The elements described below address development in landslide terrain. The process applies equally well and has been used by the authors for other types of geologic hazards. Figure 1 presents a decision tree diagram illustrating the process described below. Although some steps may be added or deleted

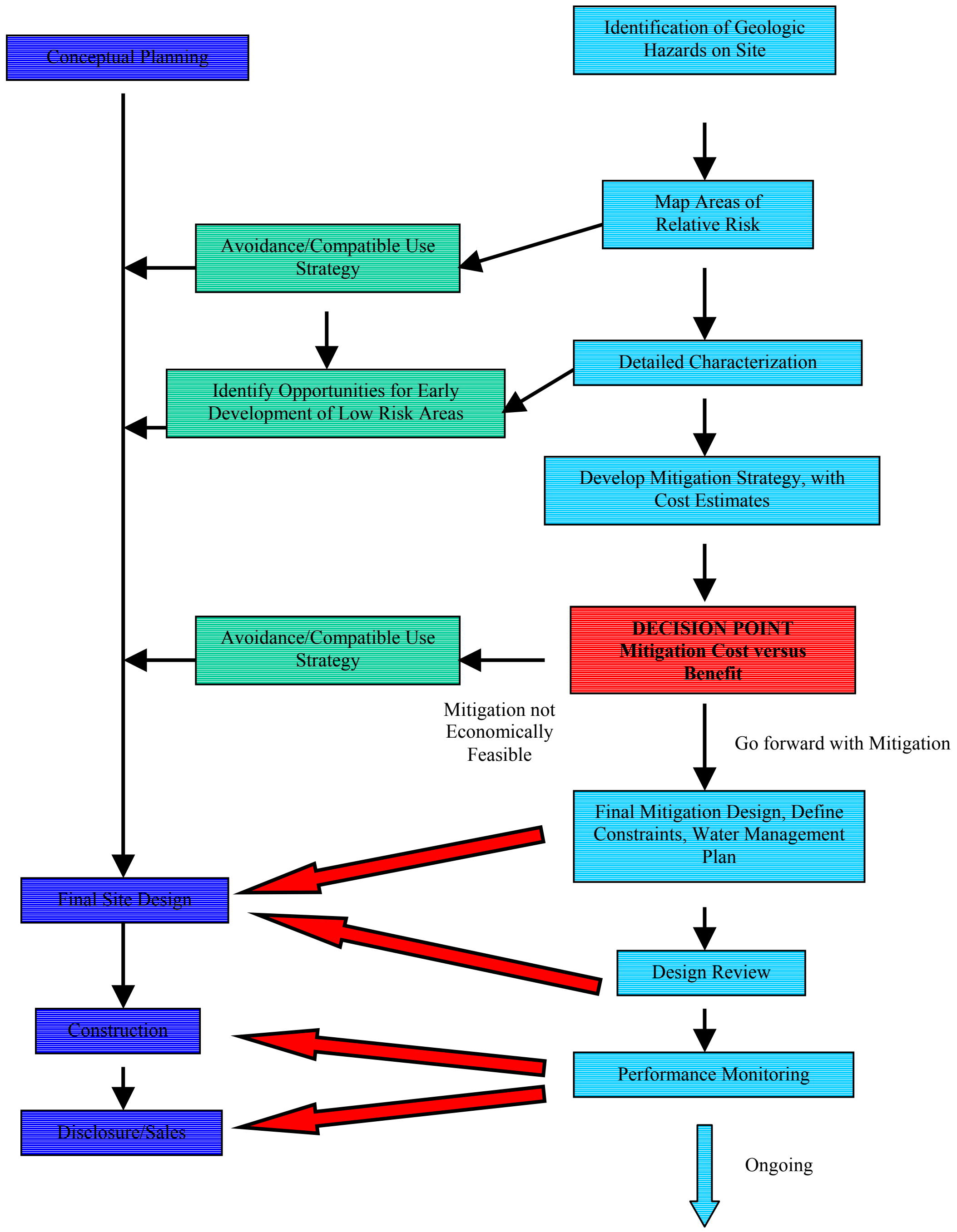

Figure 1. Decision Tree for Development of Project with Geologic Hazards

depending upon the particulars of the site and the proposed project, the diagram illustrates the general timing and sequence of tasks for addressing the geologic hazard conditions in concert with the overall land planning, approval, and design process.

Conceptual Planning Steps

Detailed Characterization **-** Traditional site characterization is generally limited to an assessment of subsurface conditions in terms of soil and rock characteristics. For properties in hazardous terrain, site characterization must focus on developing a detailed understanding of the hazards. For landslide terrain, this includes defining the dimensions of landslide masses, both at the surface and at depth, and characterizing their movement or estimating potential for movement. A clear understanding of groundwater occurrence, subsurface flows, and elevated seepage pressures must also be developed, as this is almost always a key factor in landslide movement. Periodic monitoring of groundwater levels and slope movements provide insight to seasonal effects and long-term trends. Other types of geologic hazards such as rockfall, avalanches, debris flows, swelling or collapsible soils, sinkholes or subsidence must also be assessed.

Risk Mapping - We have found that mapping relative risk over the property is very valuable to the planning process. In this exercise, a map is developed which delineates areas of relative risk, based on the absence, potential, or presence of hazardous conditions, most often expressed as areas of relatively low, moderate, or high risk. The map is based on an on-the-ground field review of the site. Examination of stereographic aerial photography is highly beneficial, as an experienced interpreter can quickly identify potential areas of concern to be checked in the field.

Develop Avoidance/Compatible Use Strategy - The map of relative risk is an extremely useful tool for site planners. Depending on the distribution of relative risk on the property, site use can be planned to take maximum advantage of lower risk areas. Areas of higher risk can be approached in a number of ways. If these areas are limited, avoidance may be the most practical approach. Higher risk areas may be suitable for recreational facilities such as open space and trails. In some cases, these areas have been successfully developed for recreational facilities such as golf courses and ski runs by incorporating mitigative measures such as lining of ponds and use of flexible piping. Scheduling of project development can also be used to advantage, developing low risk areas first and postponing development in areas where mitigation may be needed until project cash flows support the development costs.

Develop Stability Model - If development is contemplated within landslide areas, a stability model is developed to evaluate the potential for ground movements. The model is built from the information gathered from site characterization.. The result is a “working model” that will be updated as new information is gathered from monitoring. If mitigation is proposed to support development in landslide areas, the working stability model is used to evaluate mitigation strategies.

Evaluate/Develop Stabilization Strategy - Development of a stabilization strategy considers both the physical measures available to stabilize the landslide, and the costs of such measures relative to the land value. The working stability model is used to develop and evaluate conceptual alternatives for stabilization. Typically, stabilization strategies may include stability-enhancing, engineered grading of slopes, mechanical support such as tie-back anchors, lowering, or depressurizing groundwater, or a combination of these. Examining alternative strategies using the stability model allows quantification of construction elements: how many anchors, how much fill, what size drains, etc. These quantities, in turn, form the basis for estimating the costs of stabilization. If the estimated cost of stabilization measures can be economically offset by the expected return in property value, design of stabilization measures can proceed. As part of this evaluation, potential stabilization alternatives may be examined in terms of risk versus cost. Generally, higher cost measures will result in lowering of risk, and the project owner must weigh these factors in terms of the economics and potential liability of the project. Where the consequences of instability are not life-threatening or do not entail severe property damage, the owner may wish accept a greater risk of consequences to significantly reduce mitigation costs.

Consider/ Develop Water Management Strategy - As discussed above, groundwater is almost always a key factor in slope stability. A water management strategy incorporates a site-specific approach to minimizing the potential for landslide movements through control of water. Surface water is managed to minimize infiltration, by appropriate site grading, lining of ponds, and redirection of water away from critical areas. The water management strategy may also include restrictions on landscape watering, and specific placement or specialized technology in septic systems. A site-wide assessment of water balance (total inflows compared to total losses) provides a basis for developing the water management strategy, and is especially critical when considering water-intensive facilities such as golf courses or ski terrain.

Liaison with Review Agencies/Stakeholders - Most properties are subject to the review of county agencies. Many counties have established policies and regulations regarding development in areas containing geologic hazards, and many counties have maps delineating hazard areas. Depending on staffing, counties will request assistance from CGS in reviewing rezoning applications, and building and grading permits. Counties are required by the subdivision law to have CGS review preliminary plan or plat application proposals. Development in areas known or suspected to contain geologic hazards will come under scrutiny at some point of the permitting process, and the earlier these issues are addressed, the better. Development of these types of properties often requires agency reviewers to weigh approval of projects “outside the box” of their usual process. The specialty geotechnical engineer can assist the developer by working with agency reviewers, facilitating their understanding of how the proposed site use and mitigation meets the goals of their master plans, and zoning and subdivision regulations. Other stakeholders may also be involved, needing reassurance that the proposed development will not endanger their interests.

Review of Development Plans - As part of the planning and development team, the specialty geotechnical engineer can provide an internal review function as the site plan develops. This is best done interactively throughout the planning process. When needed, the specialty geotechnical engineer attends planning board meetings to support the project team on controversial elements of potential site geologic hazards.

Decision Point

During the conceptual planning process, one or more decision points may be reached. If a site has geologic hazards which significantly impact the intended site use, decisions must be made regarding the viability of the project. Such decisions may include altering the proposed site use, reducing unit density, or budgeting to construct mitigation measures. These decisions are based on the economics of cost versus benefit, related to estimated mitigation costs, expected land sales revenues, or lost revenues if the project must be scaled back to avoid hazards. If mitigation is not economically feasible, then avoidance strategy may be used to restructure the project.

Final Site Design

Define Constraints for Development, Design, and Construction - In order to reduce risk, specific development constraints may be incorporated into site planning and use. Constraints do not directly modify or remove a hazard. Instead, constraints address hazards by avoiding hazardous areas, (based on the risk mapping for example), limiting site use, or incorporating features into the site improvements to mitigate a hazard. Examples of this might be use of structural walls in the ground floor levels of homes to withstand avalanche or debris flow impacts, or limiting the heights of cuts and fills in landslide deposits. These may be in lieu of, or in addition to other site modification measures. To be effective, development constraints need to be incorporated into project permitting documents, and must carry over into the building permit process.

Review Infrastructure Design - In some cases, potential geologic hazards can be increased by the site modifications associated with development.. A geotechnical review of design elements for site infrastructure includes evaluation of cuts and fills for roads, utility locations, foundation conditions, and other site improvements to verify that the proposed designs meet stability requirements and avoid encountering or creating undesirable conditions.

Review During Construction - The specialty geotechnical engineer should be available during construction to address any concerns, which arise as foundations are excavated, cuts are opened, and unforeseen conditions are encountered. Field changes are part of any civil construction project, and appropriate responses are especially critical where geologic hazard conditions are present.

Disclosure to Owner - If property title transfer occurs as part of the project, the receiving owner is entitled, by law, to disclosure of hazardous conditions that exist on the property. As part of the transaction, the seller is required to submit site information to the buyer, and this is generally done by providing a copy of the geotechnical report(s). Buyer/owner concerns can be alleviated by having the specialty geotechnical engineer available to answer questions regarding the geologic hazard conditions and the implications to the property owner. Most commonly, the buyer/owner wants to be reassured that any existing conditions will not preclude his intended use of the property “as advertised”. Grading and development of individual residential lots, generally controlled by the receiving owner, also have the potential to exacerbate geologic hazard conditions. Specific requirements that apply to geologic hazards which are attached as a note to the final plat must be met in order to obtain approval for building or grading permits. However, disclosures to the owner should also include appropriate cautions regarding site modifications, landscape watering restrictions, and other issues related to the site-specific geologic hazard conditions. This can be done through the developer providing the receiving owner with reports and recommendations by the engineering geologist/geotechnical engineer.

Performance Monitoring - Instrumentation installed during the site characterization is generally monitored throughout project development. It is also advantageous to continue monitoring beyond construction to verify that mitigation measures and development constraints are performing as designed. In some cases, long-term monitoring is required by permit approval agreements. When problems develop, the early warning provided by monitoring can help minimize the cost of mitigation (or remediation).

Clearly, this approach can be expected to result in both higher consulting fees and higher construction costs than would be realized for a "hazard free" site. However, the authors' experiences in Eagle County in recent years suggest that the higher costs are well justified by the return of revenue from these properties. Furthermore, when dealing with development in hazardous areas, in most cases it can be easily demonstrated that on the cost of investigation, design, and mitigation during site development is much less than remediation after damage occurs.

CASE STUDIES

The following are two case studies of development projects in Eagle County. The primary geologic hazard conditions addressed at these sites are landslide terrain. The project descriptions describe the process utilized for planning, permitting, design, and construction of these projects. The various process steps described above, as used for each if these projects, are highlighted in bold type.

Red Sky Ranch, near Wolcott, Colorado

Red Sky Ranch is a major recreational/residential development nearing completion at Wolcott, Colorado. The development includes two signature golf courses and 87 residential lots. Large portions of the development lie on ancient landslide terrain and these conditions were addressed early in the planning process. A formal decision process was not applied on this project, although the interaction of the geologic hazards mitigation to the permitting and development process is summarized in Figure 2.

Detailed Characterization of the site had been started by previous investigators. These investigations had identified areas of possible active landslide movement, had defined the general limits and character of landslide deposits, and generally defined groundwater conditions. In the early stage of site planning, **Risk Mapping** was used to prepare a map of "Development Constraints". This map delineated areas of the site in terms of relative risk of future ground movements, based on a property-wide surface reconnaissance and the findings of previous investigators. From this map, the project planner used an **Avoidance/Compatible Use Strategy** to develop a site layout plan, concentrating residential lots in the areas of little or no risk. Golf course facilities and open space were planned in the areas with moderate risk. Limited portions of the east golf course were planned for high risk areas.

Since large portions of the property, including those categorized as lower risk, lie within known landslide deposits, a working **Stability Model** was developed. The purpose of the model was two-fold. First, it allowed examination of overall stability of the site to confirm that the landslide was not likely to be experiencing or on the verge of movement. Second, the results of the model assisted the permitting process by reassuring agency reviewers that development planning was based on a sound understanding of the risk. Since the landslide was determined to be stable under the existing and proposed site conditions, it was not necessary to develop a stabilization strategy.

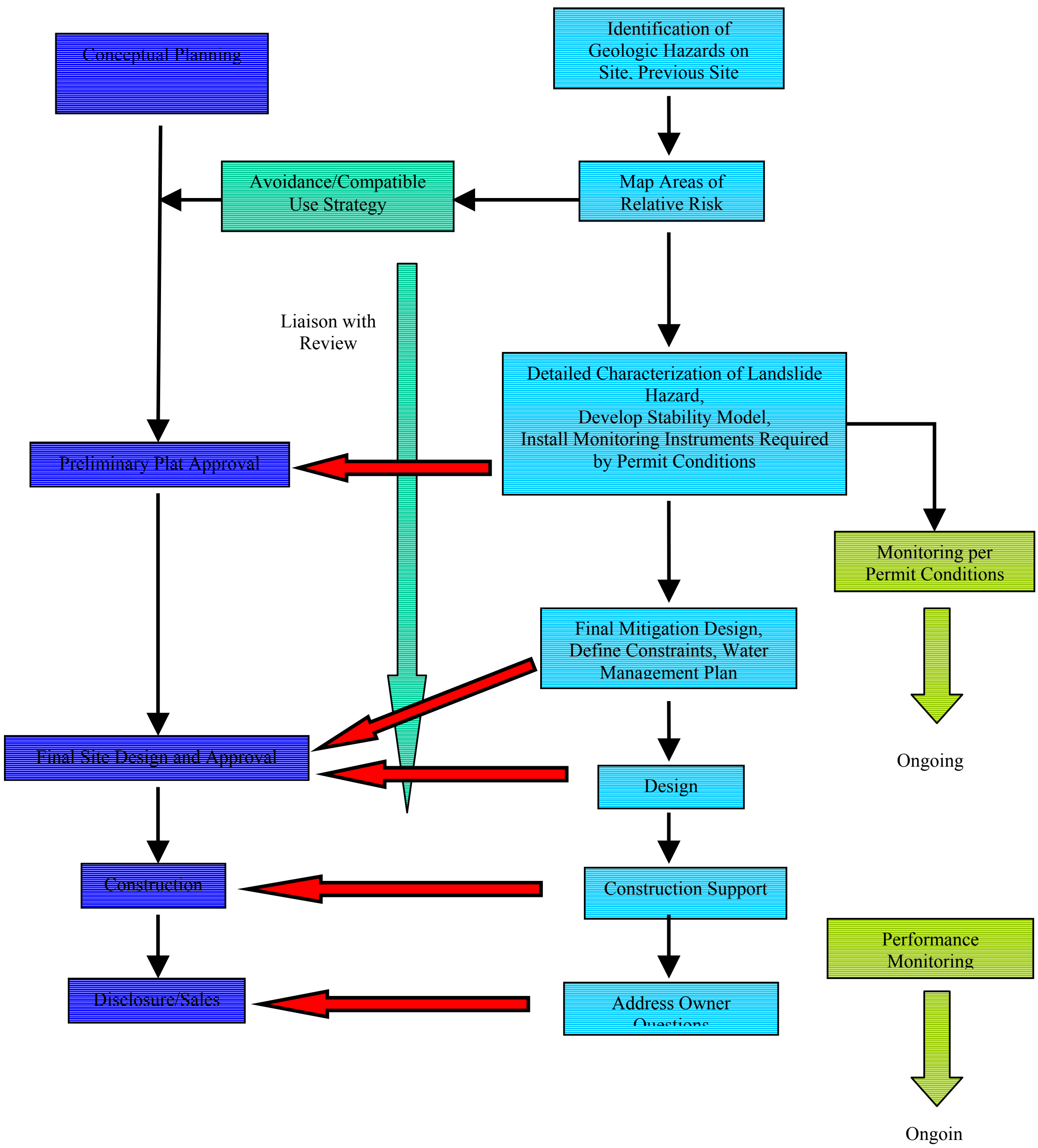

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Figure 2. Decision Tree Illustrating Process for Development of Red Sky Ranch.

During the permitting process, agency reviewers identified control of groundwater, to prevent reactivation of the landslide, to be a key requirement for development. This was of particular concern due to proposed golf course irrigation and residential septic systems. Consequently, a **Water Management Strategy** was developed for the property. The water management plan included a water balance analysis for the entire east portion of the site, considering both natural and post-development additions to the groundwater regime. The water balance showed that the golf course could be operated in such a way as to have no net increases to the overall groundwater system. Consequently, potential impacts to landslide deposits were low. The authors also worked with the golf course irrigation designer to prepare a plan for monitoring and control of irrigation during golf course operation. **Constraints for Development, Design, and Construction** included limitations on the locations for septic systems, minimizing the potential for seepage into moderate or high risk areas by using low discharge rate septic systems. Other constraints are lining of all ponds, and the use of flexible piping and utility connections where golf course infrastructure occurs within areas delineated as having higher risk of ground movements. Design constraints also apply to the steepness and height of cuts and fills in critical areas.

During the permitting process, the authors attended planning board meetings to give testimony and answer questions regarding landslide risk and mitigation on the property. They provided **Liaison with Review Agency** personnel, including CGS to address their concerns and facilitate approvals. The geotechnical specialists participated in **Review of Development Plans** in the preliminary stages of the project, then provided **Review of Infrastructure Design**. **Review During Construction** included the geotechnical specialists periodically visiting the site to assure that design assumptions were supported by actual field conditions, address unforeseen field conditions, and review the results of foundation observations. The geotechnical reports provided to buyers as part of **Disclosure to Owners** included information on the landslide issues for the property. As residential lots were sold, the authors answered buyers' questions regarding the geotechnical report and the implications for residential construction.

Monitoring of the landslide deposits began early in the site planning process during the characterization efforts. **Performance Monitoring** is ongoing, as dictated by permitting requirements. Quarterly measurements are made at a number of key locations throughout the site to check for potentially damaging slope movements and none have been indicated over the 5 years of monitoring to date.

Confidential Residential Lots, Eagle County, Colorado

Development of several residential lots in Eagle County is being considered, and the development strategy is still in progress, at the conceptual level, as of this writing. The property includes an area of active landslide movement. Over 7 years of monitoring of slope movements shows that the landslide has been and is continuing to move quite steadily. The concept of stabilizing the landslide to provide saleable lots was briefly examined previously, and was shelved as being too expensive to consider at that time. Due to escalating land values the owner wanted to revisit development options with a more detailed study.

As the decision was made to undertake further investigation of development options, a decision tree was developed to guide the process and provide logical **Decision Points** for assessing the risks and associated costs of the project. The approach included use of structured decision process to examine risk and mitigation trade-offs for the project. Figure 3 presents the decision tree prepared at the outset of the work. As the work progressed, it became apparent that development of initial lots needed to be delayed until lower risk areas identified during the preliminary site evaluation were confirmed by the results of the final site evaluation and associated monitoring. Figure 4 shows the decision tree that reflects the actual progress of the work.

Site investigation, mapping, and monitoring completed by previous investigators provided the first information for **Detailed Characterization** of the site. Additional site studies were completed with subsurface drilling and installation of monitoring instruments to characterize the landslide in greater detail and support a sound evaluation of possible mitigation measures. **Risk Mapping** was used to define the suspected limits of the active landslide. Based on the detailed site studies, a **Stability Model was Developed**. The model was used to **Evaluate Stabilization Strategies**, including tie-back anchoring, shear keys, dewatering, and combinations of these. Consideration was given to **Development of a Water Management Strategy** to lower groundwater levels at the landslide's shear surface as part of the stabilization strategy. Based on these evaluations, stabilization of the landslide was still not considered economically feasible.

Based on the risk mapping, a second phase of subsurface investigation and instrumentation was undertaken to evaluate portions of the property which appeared to be outside the active landslide, and thus have lower risk to development. The results of the investigation and subsequent monitoring results led to **Development of an Avoidance/Compatible Use Strategy**. This strategy proposes that two of the lots determined to be outside of the active landslide area be developed immediately, postponing development of the remaining lots. The project is now at the decision point as to whether or not to go forward with limited development of the site.

Along with evaluation of potential stabilization strategies, consideration was given to how **Constraints for Design and Construction** might benefit the project. Design constraints were proposed to include special design of rigid mat foundations to resist damage due to ground displacements, should they occur. Other constraints discussed were limiting the types and ownership of structures. The site studies and results of stability strategy analyses are documented to support further study in anticipation of future consideration of development, if property values continue to rise.

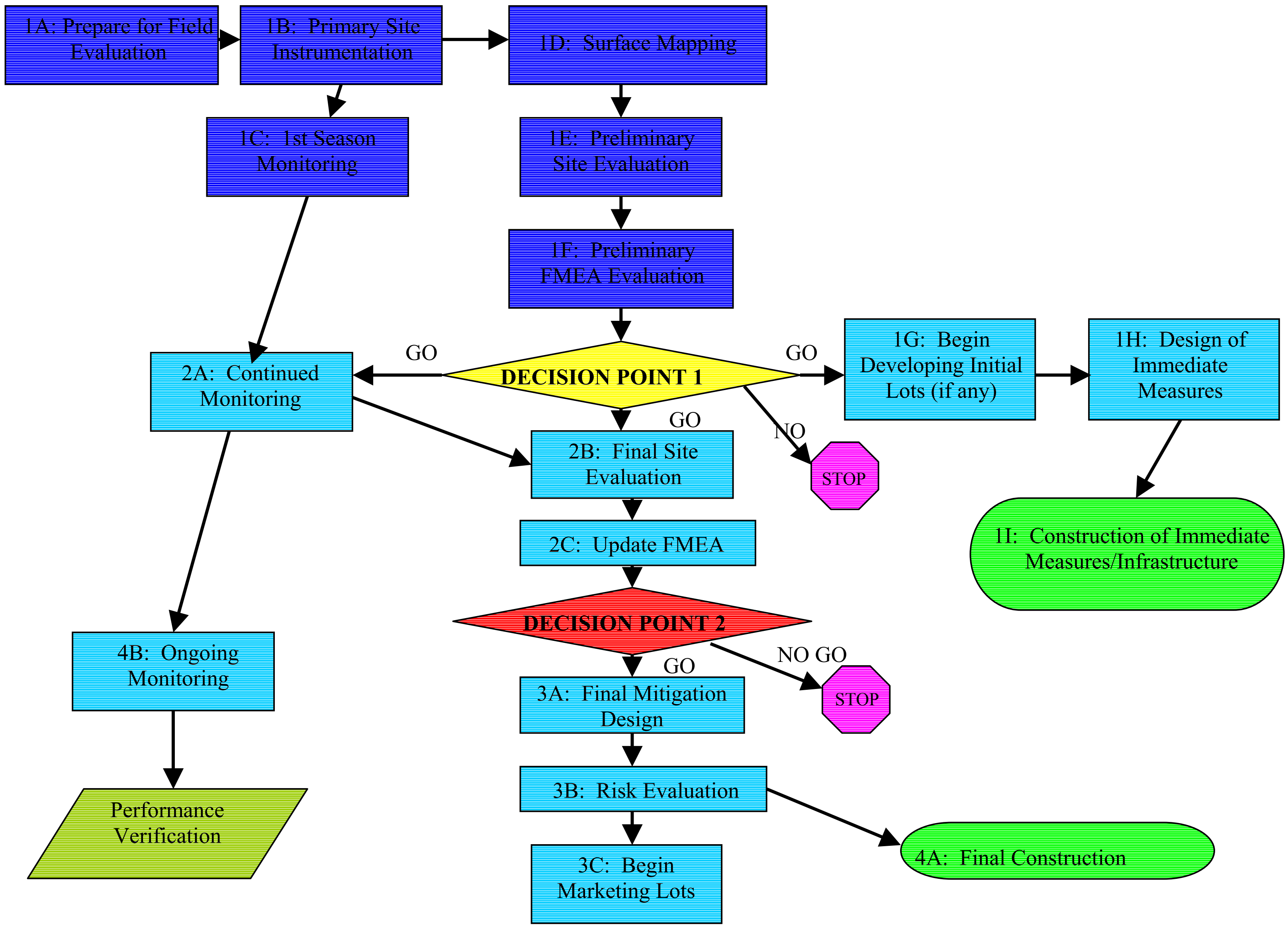

Figure 3. Decision Tree for Development of Eagle County Lots.

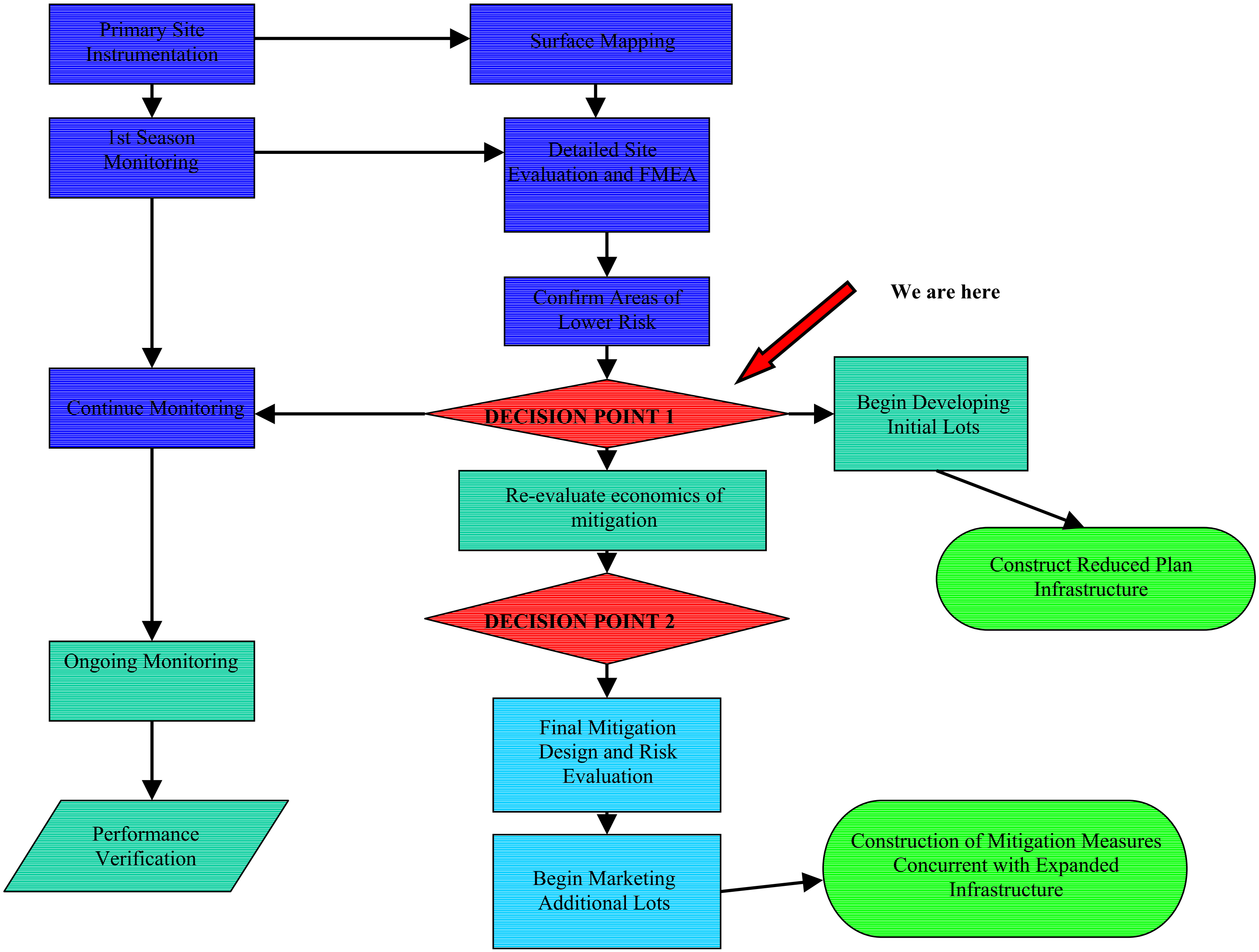

Figure 4. Actual Project Progress for Eagle County Lots.

SUMMARY

The discussion and examples given above focus on the types of geologic hazard issues and development projects that have been most common in Eagle County. The same principles and methodology apply equally well to other areas, other projects, and other types of hazardous terrain. These approaches are, for example, being used for development of property in areas of historic mining, where underground openings, waste deposits, and mine water discharges impact the site. A similar process has also been used in Eagle County for properties subject to debris flow hazards.

ACKNOWLEDGEMENTS

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COUNTY PERSPECTIVE OF GEOLOGIC HAZARDS IN LAND USE PLANNING

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Key terms: Jefferson County, geologic hazards, land use planning

ABSTRACT

Jefferson County is unique in Colorado with its varied topography and proximity to metro Denver. This position has allowed Jefferson County to experience significant growth since the 1970s, which has also increased the amount of property damage due to geologic hazards. Geologic hazards in Jefferson County range from heaving high dip bedrock, rockfalls, landslides and subsidence from historic mining.

Jefferson County has addressed geologic hazards in their regulations since 1976, with the adoption of Geologic Hazard Overlay Zone Districts. These districts addressed areas identified as slope failure complex, landslide area, rockfall area, and subsidence area. More recently, the county adopted the Dipping Bedrock Overlay District in 1995, which encompassed the area of heaving high dip bedrock. Prior to development in these districts, reports detailing the geological constraints, potential impacts on the development and mitigation measures, must be provided to the county.

Report requirements in Jefferson County vary, depending on the proposed development and planning stage. Geologic and geotechnical reports are required for a rezoning application in geologically sensitive areas, while they are not required until the preliminary platting stage in other cases. Both the Zoning Resolution and Land Development Regulation provide detailed requirements of what is required in each report.

There has been a significant amount of damage to private and public property in Jefferson County as a result of geologic hazards, which has resulted in losses of millions of dollars. The county realizes that geologic hazards must be adequately addressed early in the planning process and that they can often drive development. The current regulations allow for an increased awareness of the hazards and should minimize the potential impact on future development.

INTRODUCTION

Both the geology and topography of Jefferson County is highly variable and that combination can provide a variety of constraints on a property that may restrict development of a site. Development in the plains must take into account swelling soils, subsidence areas, and heaving bedrock, while the properties in the mountains may have to contend with slope failure and rock falls. Regardless of the geologic hazard, proper investigation, mitigation and construction must be performed in order to increase the likelihood of satisfactory performance. Unsatisfactory performance will result in, as a minimum, additional costs for the private and/or public property owners and possibly unsafe development.

A significant amount of development has occurred within the Designated Dipping Bedrock Area since 1940. The area itself contains eight sedimentary formations of the Cretaceous age, including the Graneros Shale, Greenhorn Limestone, Carlile Shale, Niobrara Formation, Pierre Shale, Fox Hills Sandstone, Laramie Formation, and parts of the Arapahoe/Denver/Dawson Formations. The western boundary of the Designated Dipping Bedrock Area is approximately the contact between the Graneros Shale and underlying Dakota Sandstone along the eastern dip slope of the Hogback ridge. The eastern boundary is approximately the eastern extent of the bedrock which dips at greater than 30 degrees from horizontal. Bedrock within the Designated Dipping Bedrock Area dips to the east or northeast at 30 to 90 degrees from horizontal.

Jefferson County has identified and published the Map Index with Known Geologic Hazards in Jefferson County. Figure 1 depicts the approximate boundaries of the Designated Dipping Bedrock Overlay District.

Prior to development in any of these areas, investigations must be performed and provided to the county for review. Both the Zoning Resolution and the Land Development Regulation provide guidance regarding what is required in the reports.

FISCAL IMPACT

Although there is not a definitive tally of costs associated to geologic hazards in Jefferson County, conservative estimates are in the tens of millions of dollars. There are countless property owners who have been saddled with costly flatwork and structural repairs due to swelling soils and/or dipping bedrock. Unfortunately, it can be very difficult and expensive to mitigate a hazard after construction and there are numerous structures that undergo structural repairs on an annual basis.

The county adopted regulations regarding the Designated Dipping Bedrock Area in 1995 and one of the last subdivisions approved prior to the regulations was the Powderhorn Subdivision near Coal Mine Avenue and Simms Street. Approximately 140 homes were built from 1994 to 1995. The homes in this subdivision were constructed without the new adopted regulations requiring a minimum of 10 feet of separation between the foundation and bedrock and were rather built on piers without sub-excavation. Based on data provided by the Building Department, over 40 building permits (approximately 30%) have been issued for structural foundation repair within this subdivision. In contrast, Home Buyers Warranty has reported the average frequency of structural repairs is approximately 1%. The average cost of a structural foundation repair is approximately $30,000.

The county has also incurred a great expense due to the hazards in both its capital improvement projects and maintenance of the existing road network. The West Coal Mine Avenue extension from South Kipling Street to Moore Street encountered two hazards, including dipping bedrock and subsidence potential from the historic Economy (Unity) Mine. Claystone bedrock was

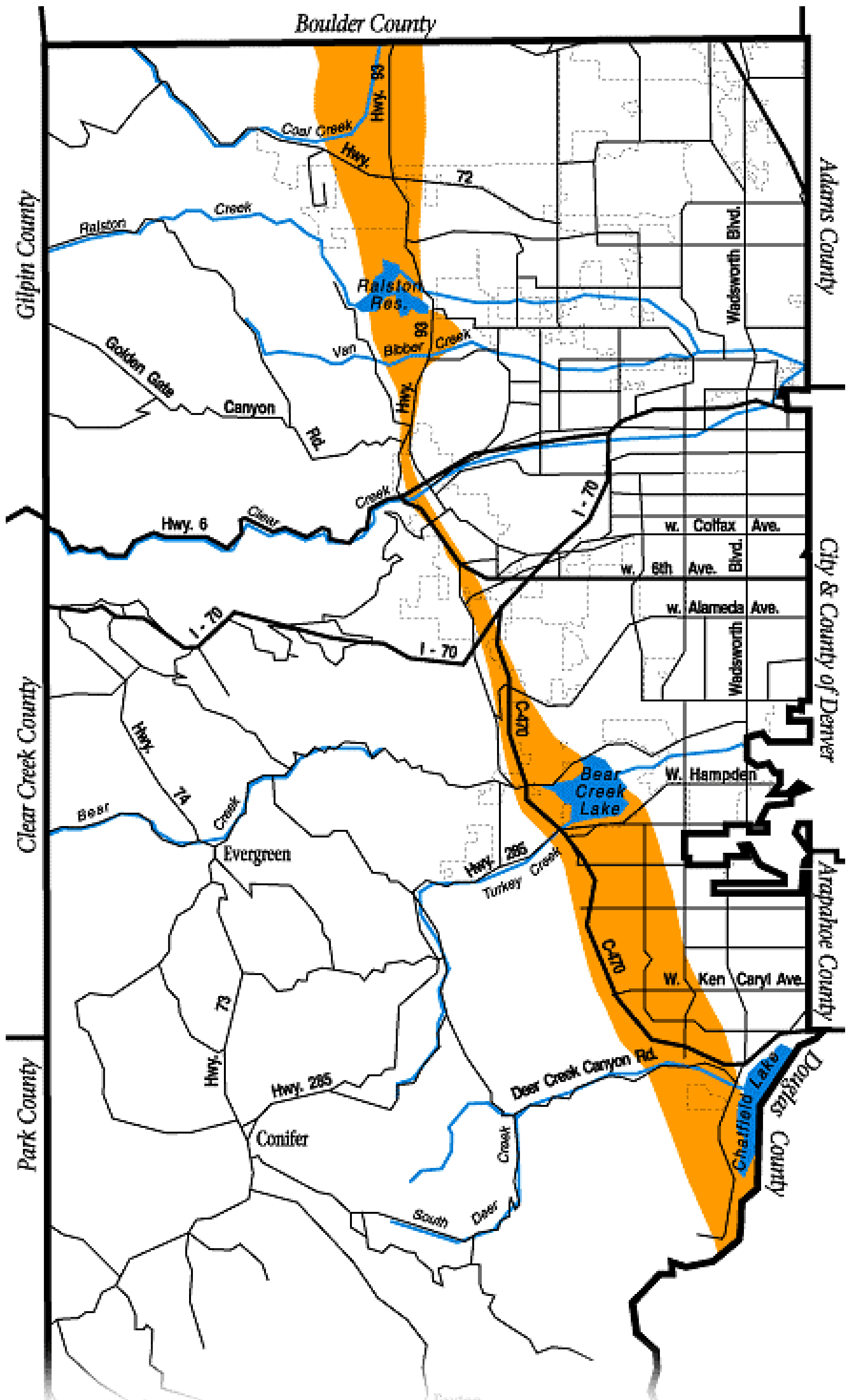

Figure 1. Approximate Boundaries of the Designated Dipping Bedrock Overlay District.

encountered as shallow as 2 feet below grade with a dip to the east at approximately 55 degrees and mine workings were reported to be within 150 feet along the proposed route. Mitigation for the hazards included overexcavation and reconditioning of the material for the shallow bedrock and a reinforced concrete span over the area that had the higher subsidence potential. Although breakout costs of the overexcavation, engineering, and reinforced slab associated with the mitigation are not available, the total costs associated with the extension of Coal Mine Avenue were $2,067,000.

REPORT REQUIREMENTS AND SITE MITIGATION

Jefferson County requires both a geologic and geotechnical report for development within the county. The requirements for the geotechnical report are fairly specific, including the minimum number and depth of borings per acreage, sampling and testing frequency, laboratory testing, and presentation. Development within the Designated Dipping Bedrock Area also requires a grading plan depicting the minimum 10 feet of separation between the foundation and top of bedrock. The requirements listed in the Zoning Resolution and Land Development Regulation, do not provide a cookbook guide, rather they are minimum standards that shall be met.

The requirements for the geologic report are also provided, however, they are not as specific, as there are several potential hazards each with varying degrees of potential risk. Since these hazards do not obey property or lot lines, they must be addressed on a larger scale. For example, residential development has occurred on historic landslide deposits and in order to minimize the potential for reactivating the landslide, the restrict the amount of moisture introduced to the subsurface, evapo-transpiration septic systems are required for this subdivision. Projects have been approved that required large-scale debris flow collection systems and rockfall mitigation measures to protect both the proposed and existing developments.

The proposed development may often change, from layout and orientation to the number of lots within subdivision, based on the outcome of the geologic report and mitigation measures. Often, in areas of higher hazards, the most efficient measure is avoidance.

CONCLUSION

Geologic hazards are obviously not a new phenomena to Jefferson County, however, given the finite amount of property and the increase in population, geologic hazards are now being encountered more frequently and mitigation may be a viable option. Data provided in the geologic and geotechnical reports often are a deciding factor if the project is feasible and the identified geologic constraints are often the driving factors of the development.

Figure 2 shows the Floyd Hill Landslide in May 1947 when approximately 35,000 cubic yards of material closed US Highway 6.

Jefferson County has strived to identify, investigate, and address geologic hazards in order to allow for proper and safe development and has improved the engineering standard of practice for the metro area with its requirements and regulations.

Figure 2. View of Floyd Hill Landslide.

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APPENDIX I

The addresses below include the Jefferson County and the Planning and Zoning Department websites, respectively.

www.jeffco.us

www.planning.jeffco.us