ENGINEERING GEOLOGY 14

Collapsible Soils in Colorado

By Jonathan L. White
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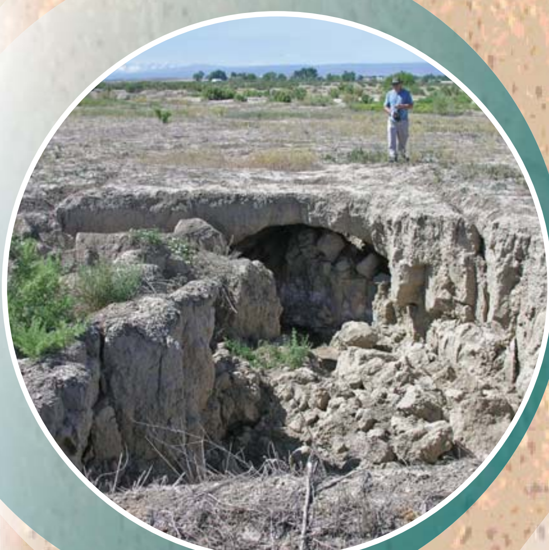

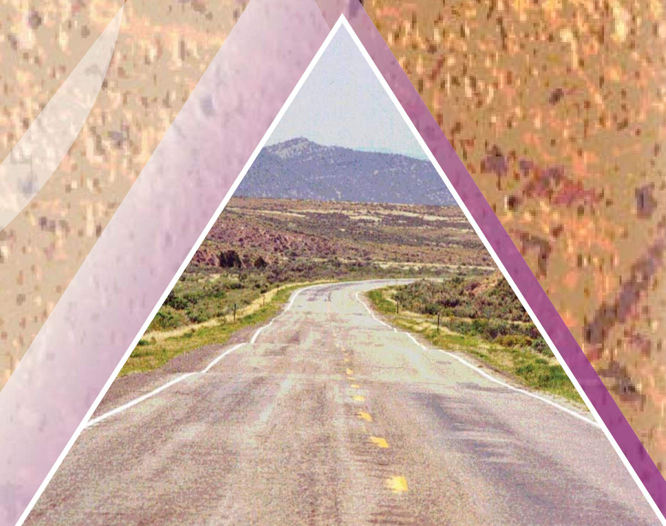

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COLORADO
DEPARTMENT OF NATURAL RESOURCES

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FOREWORD

Colorado Geological Survey Engineering Bulletin 14, *Collapsible Soils in Colorado*, describes the geologic setting, the geomorphic and soil conditions, locations of potential susceptibility, and engineering properties of collapsible soils in Colorado. This bulletin is the result of a comprehensive and multi-year effort to understand collapsing soil behavior and the geologic and geomorphic conditions where they can form. In addition, this report contains a 1:1,000,000-scale map of Colorado that shows locations of soil collapse compiled from soil test data and damage incidents, climatic exclusions zones, and areas of the state where collapse-prone soil may exist. Jon White was the program manager and wrote this report with assistance from co-author, Celia Greenman. Mary Eberle of Boulder, Colorado edited the final manuscript. The objective of the CGS collapsible soil program is to increase the public and professional awareness of collapsible soils in Colorado; and to improve the existing geological and geotechnical professional standard-of-practice through the discussion and evaluation of the tools and techniques that are available to investigate and assess what is a widespread geologic hazard, common in almost all semi-arid non-mountainous areas of Colorado.

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ABSTRACT

Collapsible soils are broadly defined as soils that rapidly settle when exposed to water. These soils are a significant geologic hazard in Colorado and other Western States of the United States having semiarid to arid climates. The collapse can occur under the weight of the soil alone (overburden pressure) or under the additional load of a building or other structure. Most collapse occurs through mechanical means where dry, low-density, high-porosity soil becomes denser when the soil-particle binding agents weaken or break after wetting. The destruction and recompaction of the soil structure under moister conditions cause settlement of the ground surface. Because the introduction of water brings about such collapse, the terms "hydrocompactive" and "hydrocompressible" are commonly used to describe collapsible soils. Other processes of ground subsidence and collapse occur in dispersive or erodible soils through (1) suspension and removal of particles by flowing water (soil piping and pseudokarst formation) and (2) actual chemical dissolution of the soil matrix in gypsiferous soils and soils derived from evaporite bedrock.

Collapsible soil forms in specific, geologically recent (Holocene) sediments that have been deposited in arid to semiarid environments. The deposits include (1) hillside gravity and slope-wash deposits, called colluvium; (2) accumulations of rapidly deposited, unsorted, water-borne mud in alluvial and debris-flow fans; (3) aggraded overbank deposits, called alluvium (silt and clay recently laid along tributary streams, flood plains, and gently sloped mud flats); and (4) windblown deposits of dust, silt, and fine-grained sand, called loess. Where soil collapse exists, an open and inherently unstable skeletal fabric characterizes the soil structure of these sediments. The common factor in the water-laid sediments is rapid deposition. In the generally arid climate, wet sediments quickly desiccate (dry out) in their original condition, without the benefit of further reworking to pack the sediment grains. Locally, ground-water levels generally never rise into these mantles of soil, which can remain unsaturated until land development. During and after development, moisture can be introduced to the subsurface soil through field irrigation, lawn and landscaping irrigation, capillary action under impervious slabs, leaking or broken water and sewer lines, and altered surface and subsurface drainage.

Soil collapse adversely affects land use and can be quite destructive to foundations, roadways, buried utility lines, septic systems, and water diversion and retention structures (canals, irrigation ditches, and dams). Severe distress, expensive repairs, and condemnation of structures have generated public interest in, and research on, the occurrences of collapsible soils; their engineering properties; and the availability and merit of various mitigation techniques used for them, not only in the Western United States but also in other semiarid parts of the world.

In regions containing collapsible soils, it is important that the geomorphology and surficial soil deposits be accurately mapped and that site-specific geotechnical investigations be completed so that structures can be appropriately designed. Mitigation of the potential hazard at the time of construction is always less expensive than future remedial repair work. Water and drainage management are important not only for new construction but also for maintenance of existing structures, which might have been designed without knowledge or consideration of collapsible-soil hazards.

1. INTRODUCTION

Collapsible soils are distinct in their ability to compact, settle, or disperse naturally when moisture is added. In many circumstances, the settlement is mechanical in nature, which can be relatively rapid such that the ground seems to "collapse." In parts of Colorado, as with much of the Western United States and other areas with semiarid climates, collapsible soils can adversely affect engineered works. Settlement from collapsible soils is seemingly the opposite behavior of swelling soils. Actually, completely different mechanisms are involved.

Soil swelling typically is caused by a mineralogical phenomenon in which clay minerals enlarge by taking water molecules into their crystalline lattice at the atomic level, which widens the microscopic clay platelets, causes the soil to expand, and forcibly heaves the ground surface (Noe and others, 1997). The opposite is also true. If moist swelling soils dry out, water molecules exit the crystalline lattice, the clay platelets shrink, and the soil contracts, which is why swelling soils are commonly referred to as shrink/swell soils.

Mechanical soil collapse and the resulting ground settlement also occur by the introduction of water, but the moisture instead weakens the bonds of the soil grains, which causes the porous soil skeletal fabric to break such that the soil grains realign into a denser configuration. The soil quickly compresses, and the ground surface subsides.

Property impacts and damage from soil collapse and settlement have been documented in Colorado ever since early settlers moved into the territory in the late 1800s (Paddock and Whipple, 1910). However, it was only in the early 1970s that damage from settlement of collapsible soils began to be examined in depth. The Colorado Geological Survey formally defined the phenomenon in Rogers and others (1974), and collapsible soils were listed as a subsidence hazard in the geologic hazard guidelines introduced in House Bill 1041 [CRS 24-65.1-103 (10), Colorado Revised Statutes, 2007] that was enacted by the Colorado Legislature in 1974. Today, as the population of Colorado increases, more terrain covered by collapsible soils is being considered for development. Despite the library of technical information and case histories, the hazard is not widely recognized by the public and land-use planners. Even among those who are aware of the hazard, there are some common misconceptions (shown in **Table 1-1**) concerning geotechnical evaluations and land-use in collapsible-susceptible soil terrains.

Table 1-1. Common misconceptions concerning geotechnical evaluations in terrain susceptible to collapsible soils.

MISCONCEPTION	POTENTIAL REALITY
1. This area is known for swelling soils, so I don't need to worry about settlement from collapsible soils.	1. Settlement can still occur. Collapsible soils and swelling soils could be present present on the same site, and a soil could be both collapsible and expansive, depending on the soil fabric, swelling-clay minerals present, and applied load.
2. A subsurface investigation that analyzes soils only to the foundation depth is adequate.	2. Where collapsible soils are present, the thickness of the collapse-susceptible soil must be known so that maximum potential settlement is determined and mitigation can proceed accordingly.
3. Geotechnical engineering tests show no collapse or only low collapse potential in the soil; there is no need to be concerned.	3. Reliance on test results should be tempered with the understanding of their limitations, especially when the soil depositional environment is well known to produce collapse-prone soils.
4. My geotechnical consultant says that as long as the site is kept dry, there will be no problem.	4. Despite the best intentions, soil-moisture contents increase under impermeable slabs, water lines break, and changes in surface and subsurface drainage do occur.
5. The site has been flood-irrigated in the past. The soils have already been wetted, so there is no longer a potential for collapse.	5. Irrigation wetting is rarely consistent in the subsurface; it can initiate soil processes that can increase the void space in the subsurface, and soils can still collapse under additional loads of a structure or pavement.
6. My home was once damaged but has been fixed so the hazard is gone. I don't have to worry about it any longer.	6. If the collapsible soil is thick and if the repair was merely cosmetic, or the foundation remediation did not go deep enough, additional wetting or migration of moisture into dry, uncollapsed layers could cause additional settlement and damage in the future.
7. My house sits on bedrock; therefore, there is no potential for collapse.	7. Certain formational and fracture-fill materials can dissolve, causing collapse and long-term settlement of "bedrock."
8. This site has been approved by the local government; it's OK!	8. Buyer beware! Sites can be approved without consideration of the geology and soil conditions or with a poor interpretation of the geologic conditions.

This publication discusses collapsible soils with respect to occurrences in Colorado. It is based on a compilation of previous research studies; susceptibility mapping; reports from mitigation and remedial construction projects completed in Utah, Arizona, New Mexico, Colorado, Wyoming, and California; and an extensive database of Colorado case histories that have been compiled at the CGS as part of a Colorado Statewide Collapsible Soil Study. The case histories were generated by (1) researching Colorado-specific information from publications, from Federal and State agencies, or from geotechnical and structural engineering companies; (2) interviewing soil scientists, geologists, engineering consultants, planners, personnel from construction firms that do remedial work, and private individuals with collapsible-soil problems; and (3) researching consultant reports archived in CGS land-use files. The data collected included geographic location, bedrock geology, surficial soil type, geomorphology, climate, and soil engineering properties on each site. **Plate 1** reveals the location of the case histories and geologic and climatic conditions that are amenable to the formation of collapsible soils in Colorado. The information in this publication could also be used as a reference for susceptible areas outside of Colorado.

2. PURPOSE OF THIS STUDY

The Colorado Geological Survey created this publication to be used by a range of people who might deal with collapsible soils: planners and government personnel who formulate and implement land-use plans and regulations; property and home owners who might have real estate where collapsible soils occur; builders and landscapers; and students, in-training geologists and engineers, and practicing professionals.

Overall, this publication is part of a collapsible-soil program at CGS that is meant to increase the awareness of collapsible soils and identify the areas of Colorado where they are potentially present. The specific objectives are:

- to explain the basic soil engineering properties of collapsible soils,
- to discuss the tools and techniques available for proper identification and characterization of the soils,
- to present the basic geologic and geomorpho logic processes that are responsible for collapsible-soil formation,
- to describe the proper level of geologic and geotechnical investigations that are needed for development planning in collapsible-soil terrains,
- to present the mitigation techniques that are currently available for new construction or for repair of existing distressed structures in collapsible-soil terrains, and
- to explain what the homeowner or landowner can do to minimize the threat of soil collapse that could result in settlement and damage.

Future activities of this program are the development of collapsible-soil hazard-susceptibility maps of selected areas of Colorado. Existing and proposed map areas are shown in plate 1.

3. WHAT ARE COLLAPSIBLE SOILS?

Collapsible soils are known as hydrocompactive, hydrocompressive, metastable, low-density, and water-sensitive soils. Most collapsible soils are dry, fine-grained soils with a honeycomb skeletal fabric of open pores that are visible both microscopically and macroscopically (visible to the eye). The soil-binding agents are relatively strong in a low-moisture state and are able to support not only the load of the overburden pressures, but also additional loads, such as buildings or embankments. However, upon wetting, a water-content threshold is reached at which the soil-binding agents weaken. The same stresses that the soil had previously accommodated now cause the soil grains to shear against each other and pack into a denser configuration that reduces the void space. The result is settlement at the ground surface, as conceptualized in the illustration shown in **figure 3-1**. The process that leads to soil collapse includes three necessary components, as described by Barden and others (1973): (1) soil composed of an open, potentially unstable structure; (2) an applied stress component that is large enough to develop a metastable condition, and (3) a suitably strong soil-binding agent to hold and stabilize the soil-grain contacts in their original metastable orientation. This process pertains to classical mechanical collapse of soils upon wetting (i.e., hydrocompaction). Soil collapse through dispersion, piping erosion, and soil-mass loss by dissolution are discussed later in the chapter.

CLASSICAL MECHANICAL COLLAPSE

Hydrocompactive soil was defined as "unsaturated soil that goes through a radical rearrangement of particles and great loss of volume upon wetting with or without additional loading" in early work by Sultan (1969). The common features of collapsible soils are (1) open structure, (2) high void ratio, (3) low dry density, (4) high porosity, (5) geologically young or recently altered deposits, (6) high sensitivity, and (7) low inter-particle bond strength (Rogers, 1995).

Soil structure is the sum of various factors, including the degree and type of aggregations, the particle gradation, the porosity, and the geometric arrangement or soil fabric of the individual soil grains. In collapsible soils, an open skeletal fabric is pronounced, and the grain-to-grain contacts are tenuous. The void space between soil grains of silt or clay can typically be seen only with a microscope. Collins and McGown (1974) reviewed some of the early soil-mechanics work in which clay-soil micro-fabric was first described as a "honeycomb" or "house of cards" structure. They discussed at length how agglomerations or aggregations of clay platelets act as individual units or grains within the microfabric of the soil. This porous microfabric can be best seen in scanning electron microscope (SEM) images. **Figure 3-2** shows SEM images of collapsible soils from southwestern Colorado (Luehring, 1988).

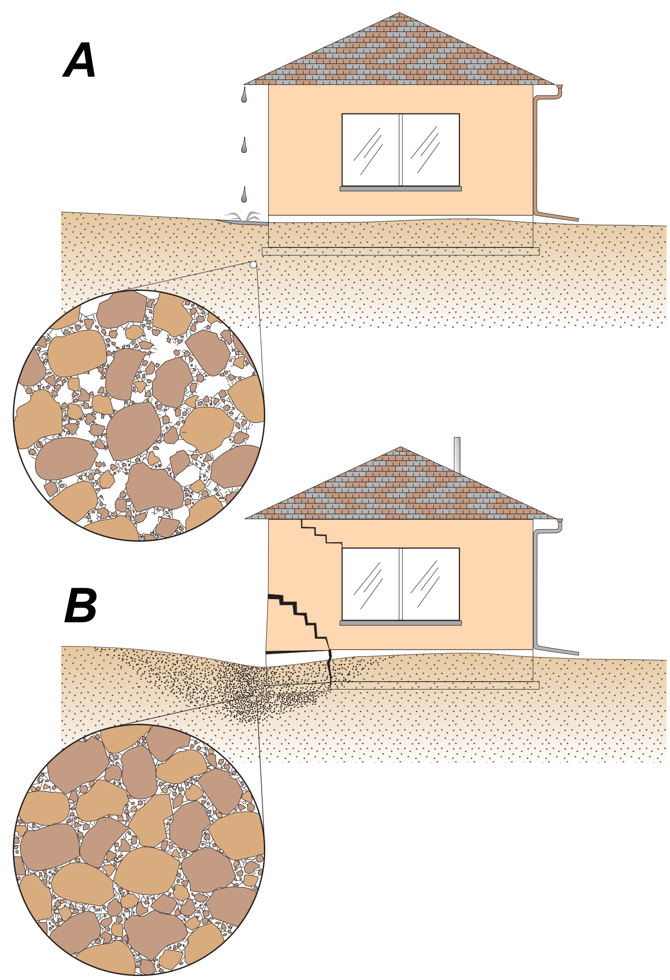
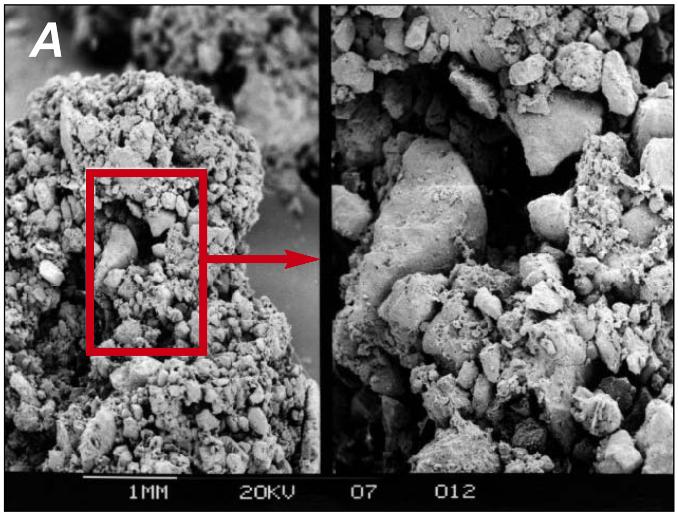
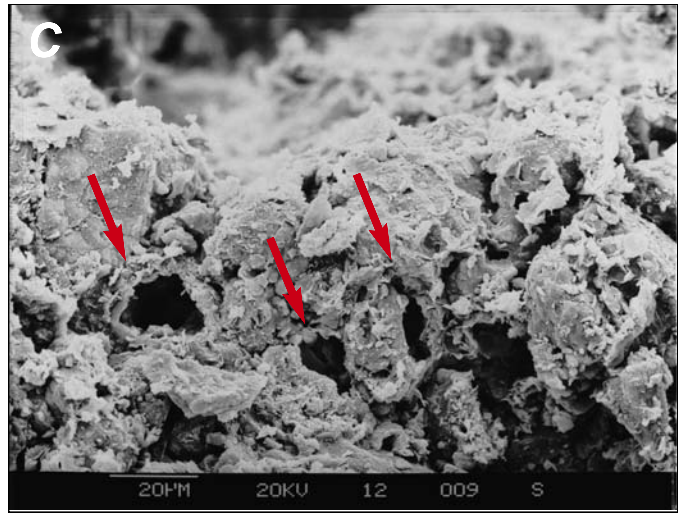


Figure 3-1 ***A*****, Wetting of high-void-space hydrocompactive soil has caused;** ***B*****, collapse and densification of the soil fabric, ground settlement, and distress to the structure.**





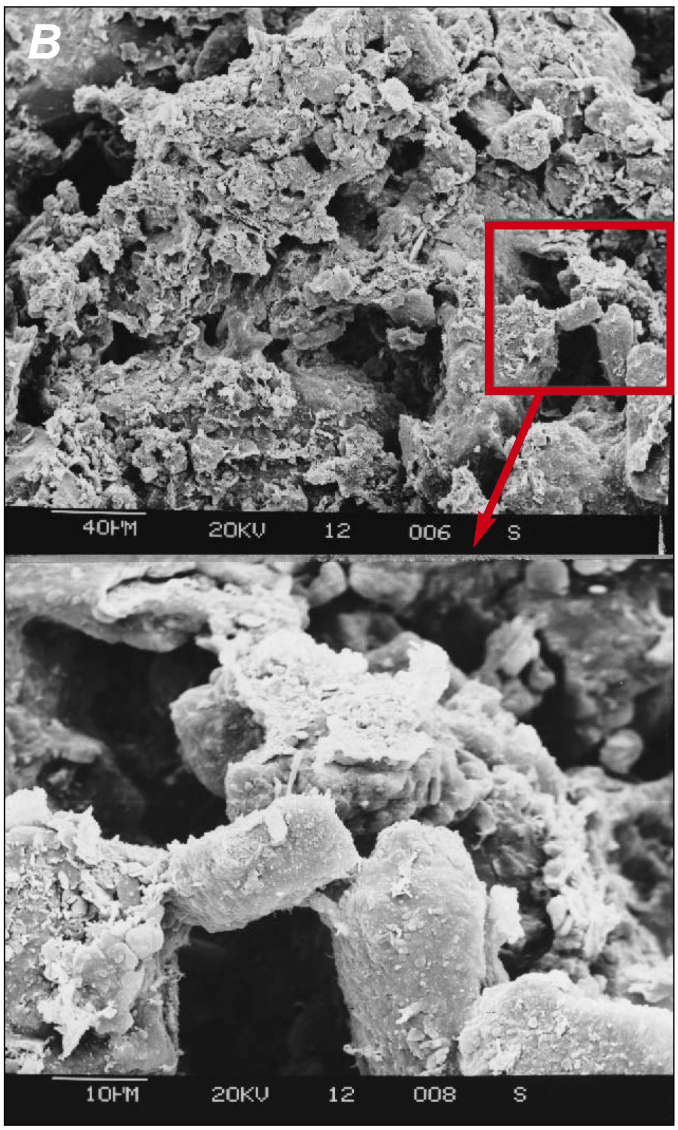


Figure 3-2. SEM images of collapsible clayey-silt soil. *A* and *B*, Void space is abundant between discrete soil grains. *C*, Clay-encrusted "shells" (arrows) line void pockets, and fragile bonds bridge between grains. Scale bar for *A (left)* is in millimeters. Scale bars for *B* and *C* are in micrometers (1 μm = 0.0001 cm). By comparison, 1/32nd of an inch is 795 μm, and the no. 200 sieve openings (upper gradational boundary of silt) are 75 μm. Boxes indicate close-up views shown by arrows. Images courtesy of R. Luehring.

Porosity or void space can also be macroscopic, visible as tiny pores in the soil sample (**figure 3-3**). These macroscopic voids can be primary, that is, formed as the geologically recent sediment was deposited. Primary voids represent either large air bubbles entrained in a "frothy" muddy debris flow that quickly consolidated or sites previously holding organic matter that has decomposed. Secondary voids form later from animal burrows, vegetation roots, or selective dissolution and removal of soluble soil constituents in the soil.

Soil that contains high percentages of gravel- and cobble-sized rock fragments can also be collapsible; such soil is termed "collapsible gravel." During the deposition, the larger rocks are dispersed in and supported by the finer-grained soil matrix, which exhibits the collapsible-soil structure. The larger rocks are not touching each other as they would, for example, in a tightly packed gravel. Typical gradations for collapsible gravels are provided by Rollins and others (1994).

One of the criteria for collapsible soils is the presence of strong, but sensitive, soil-binding or cementing agents to hold the soil fabric in its initially open configuration, which becomes unstable upon wetting and then fails such that the soil structure loses void space (Barden and others, 1973). There are several types of water-sensitive, soil-fabric binding agents that have been found to soften, disperse, reduce bond pressures, or dissolve when wetted (Jennings and Knight, 1957; Bull, 1964; Dudley, 1970; Barden and others, 1973; Clemence and Finbarr, 1981). The major ones are:

- capillary tension—also called soil suction, which when high improves soil shear strength at lower moisture contents because the water meniscus that bridges soil particles becomes smaller and holds the soil grains tighter as the soil dries after its initial deposition,
- silt bridges (also utilizing capillary tension),
- clay bonds (clay bridges of clay agglomera tions and flocculated clay that buttress silt and sand grains), and
- chemical precipitate—either carbonate or sulfate (gypsum)—as a cementing agent.

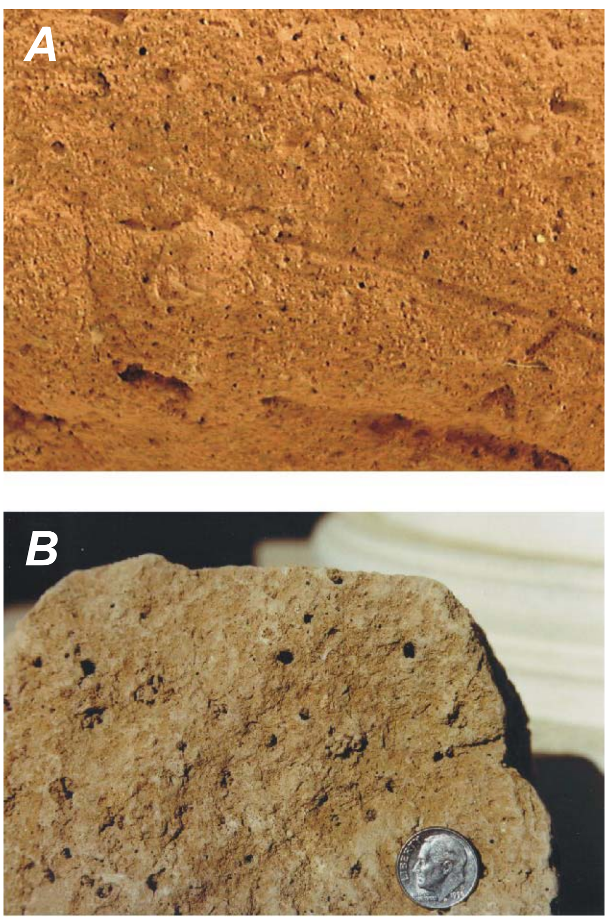


Figure 3-3. Visible, macroscopic voids in collapsible soils. *A*, A sandy-silt soil in which the primary voids were created at the time of deposition. *B*, A more clayey soil with secondary voids formed by microdissolution and biogenic activity.

The soil-binding agents can be quite strong, but weaken quickly in the presence of water. Houston and others (1988) and Beckwith and Hansen (1989) described the agglomeration of sand, silt, and clay grains as being "tack-welded" in the loose honeycombed state by the various binding agents. Upon wetting, the "tack-welds" fail quickly, clay agglomerations disperse, and grains shift so quickly that the honeycomb soil fabric "collapses"; hence the term "collapsible soil" describes any soil that is vulnerable to this process. Figure 3-1 illustrates this phenomenon at the microscopic level as well as how the phenomenon is manifested at the ground surface as subsidence and settlement. This condition is diametrically different from normal soil consolidation in the engineering sense. For clay- or silt-rich soils, consolidation implies expulsion of water and lowering of the soils' intergranular pore pressure. Collapsible soils, on the other hand, absorb water as they settle and compact, which is why "hydro-" (denoting water) is used as a prefix in the terms "hydrocompaction" and "hydrocompression." Chapter 4 provides information on engineering properties.

Hydrocompaction results in a conical-shaped zone of subsidence in the soil layer (fig. 3-1*B*). The perimeter of the settlement depression may show arcuate ground cracks, which define the wetting front where the compacted and subsiding soil has pulled away from the unaltered ground surface **(figure 3-4)**. These depressions can capture additional runoff that can cause further wetting and progressive subsidence. This mechanism is the predominant means of soil collapse for most low-plasticity soils with higher silt and sand content that are formed in eolian, alluvial-fan, and hillside colluvial environments.

OTHER METHODS OF SOIL COLLAPSE

Dispersion and Piping Collapse

In another type of soil collapse, soil mass can be lost by processes that physically remove sediment grains, primarily through dispersion and piping erosion. Overbank alluvial soils with high percentages of clay and silt and high salt contents can be dispersive and prone to piping. In many arid locations covered with recently-deposited clayey-silt alluvium, pedogenic gypsum soil horizons and heavy concentrations of gypsum and (or) other salts may occur. Such soils are commonly dissected by U-shaped gullies and arroyos typical of erosion in arid environments.

Dispersiveness is the property that causes colloidal suspension of clay particles in the presence of fresh water. The soil mass and volume reduce as soil disperses, particle by particle, in the water, which causes the water to become cloudy. Dispersion is also

Figure 3-4. Ground settlement during a soil-flooding test near Rifle. Hydrocompaction has created arcuate, concentric ground cracks in the 10-ft-high test embankment adjacent to the pond (see arrow). Note collapse features in floor of pond due to piping erosion. Photo courtesy of R. Barrett and CDOT.

referred to as colloidal erodibility. Dispersion of clay and silt particles widens soil fissures and creates subsurface channels called pipes. Dispersive soils can appear dense and have a high clay content; they do not necessarily have the appearance of a loose, highly erodible soil.

Piping erosion (i.e., formation of subsurface soil pipes) results when water is able to flow through sediments in subsurface channels. Piping passageways can begin at holes or fissures formed from several methods: the decay of plant roots, animal burrows, eluviation (i.e., the passage of silt and clay grains suspended in water through the interstices or connected pore spaces of a soil), dissolution of soluble constituents of the soil, desiccation cracks from swelling and shrinking of clay-rich soil, and subsidence cracks. Because the ground surface is often relatively impermeable, runoff is directed down these vertical macropores (cracks), and erosion is transferred underground where large pipes and wide fissures form. Piping is a common type of erosion of clay- and silt-rich soils in dry lands (Parker and Higgins, 1990) and thus is widespread in the Eastern Plains, Colorado Piedmont, and Western Slope plateau regions of Colorado.

Parker and Jenne (1967) described soil pipes in association with soil stress cracks or fissures formed from subsidence related to both hydrocompaction and ground-water removal that is common in Arizona, Nevada, and California. Parker and Higgins (1990) made the following generalizations about places where soil dispersion and formation of soil pipes may occur: (1) there is enough water to fill soil cracks and flow through the soil pipes, (2) the soil has shrink-swell clay-mineral constituents (e.g., higher smectite clay percentages), (3) the soil is generally dry or desiccates thoroughly on a seasonal basis, and (4) there is a topographically low outlet or discharge point for the water flow, such as an arroyo, ditch, or cut slope. Parker and Higgins also wrote that rates of piping erosion are enhanced by high percentages of exchangeable sodium ions and instability of the clay-agglomerated soil grains (on the micro level) and minimal vegetation cover and low slopes (on the macro level), which could increase infiltration and flows toward areas of shallow subsidence. These soils are generally unsaturated, fine grained (clay and silt), and low density, and they have moderate to good shear and bearing strengths when in a dry state. Soil collapse occurs by soil-mass loss (i.e., enlargement of subsurface pipes and voids and subsidence and failure of the bridged material into the void).

After connected pipes form an outlet at an arroyo, further piping erosion can also occur by corrasion (i.e., tunnel scour), which is the frictional wearing away of soil by the mechanical action of turbulent water with suspended sediment (Parker and Higgins, 1990). This action accelerates the enlargement of pipes and voids and can overtake dispersion as the primary erosive tool. These pipes and subsurface voids can be quite extensive, even cave-like, and can widen and migrate laterally through the subsurface over time, with no surface expression. At some point, failure of the soil bridges above the voids and fissures leads to the spontaneous appearance of modest-sized sinkholes and troughs. Livestock and farm equipment have been reported to fall into voids when soil bridges at the surface failed abruptly. **Figure 3-5** illustrates the complexity of forms of structural failure that can occur (Parker and Jenne, 1967).

The landforms resulting from soil dispersion and piping are called “pseudokarst” **(figure 3-6)**. This term, coined at the beginning of the twentieth century, is a variation of “karst,” a term derived from the geographical name of part of Slovenia in southern Europe that contains terrain characterized by open voids, caverns, subterranean flows, sinkholes, etc., formed by the dissolution of limestone. Whereas true karst features result from the molecule-by-molecule dissolution of soluble rock, pseudokarst features are primarily a result of grain-by-grain removal of soil or poorly cemented bedrock constituents (clay, silt, and very fine grained sand particles) by suspension in moving water.

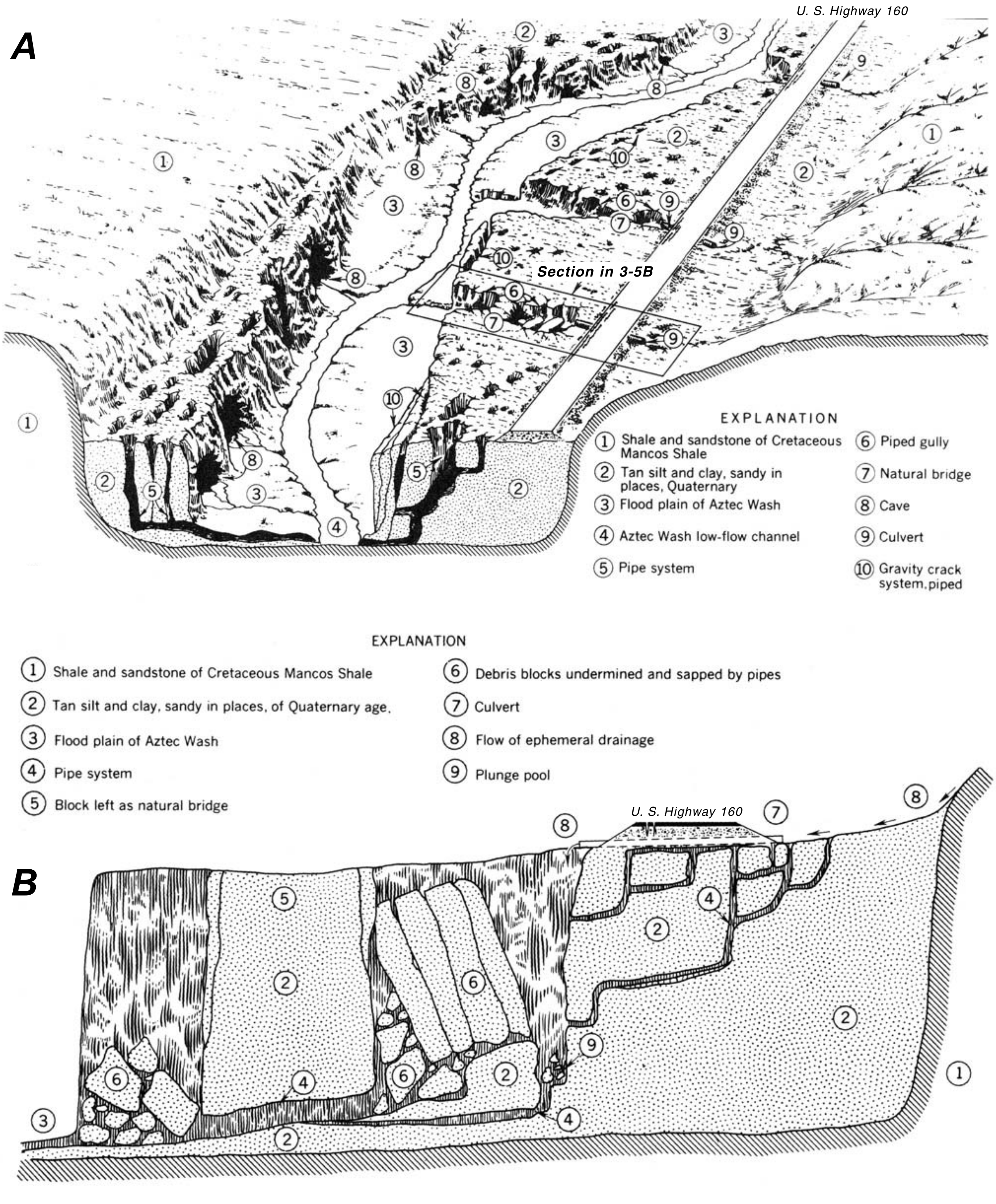


Figure 3-5. ***A*****, Terrain resulting from piping dissolution of geologically recent alluvial sediments and the creation of pseudokarst morphology in an example along U.S. Highway 160 at Aztec Wash in southwest Colorado.** ***B*****, Cross section through the terrain. Illustration from Parker and Jenne (1967).**

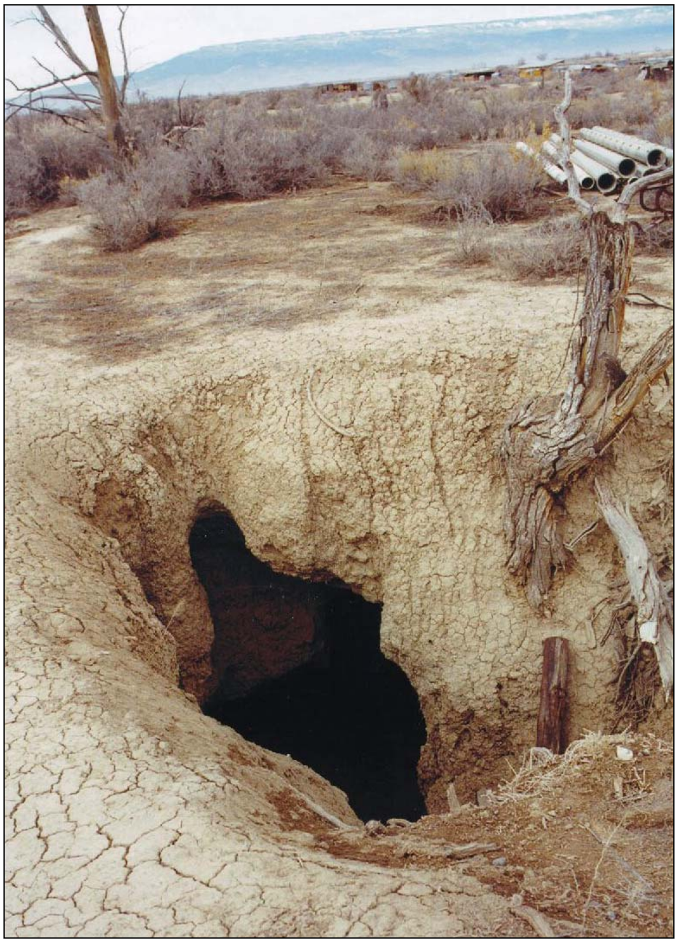


Figure 3-6. Sinkhole and cavern formation due to soil dispersion, piping, and void collapse along Loutsenhizer Arroyo, north of Montrose.

Dissolution of Soluble Constituents

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 this type of dissolution occurs where evaporite bedrock is exposed near the surface and where soils contain significant percentages of pedogenic gypsum, either dispersed in the soil or as discrete Bk-By soil horizons. Any exposure to water can cause dissolution of the gypsum. This process, although not triggering rapid collapse, can cause long-term subsidence and settlement at the surface. Soil scientists are aware of the problems with these types of soils (Nettleton and others, 1982), and well-known occurrences exist in the Colorado Four Corners region (Doug Ramsey, personal communs., 2001, 2004). The Soil Survey of Aspen-Gypsum Area, which describes a region where Eagle Valley Evaporite bedrock is widespread and exposed at the surface, also cautions about building on potentially settling gypsiferous soils (Alstatt and Moreland, 1992). A more in-depth description of the environments where these soils are found is included in Chapter 5.

DAMAGE THAT RESULTS FROM SOIL COLLAPSE

All types of collapse, regardless of genesis, result in subsidence and settlement of the ground. Severe subsidence could cause adverse land impacts; if the strain of differential settlement exceeds the strength of a structure foundation, utility line, or pavement, then structural distress and damage might occur. Examples of land impacts and structural damage are shown in **figure 3-7**.

Figure 3-7. *A*, Typical damage to foundation walls and brickwork from settlement is visible on a school in Canon City.

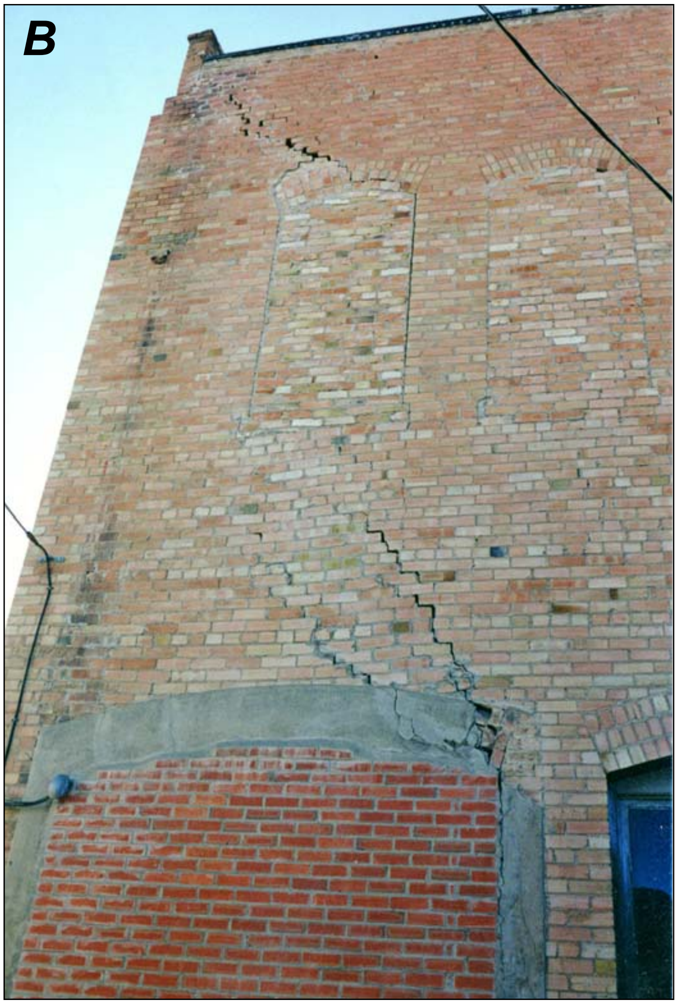


Figure 3-7. ***B*****, Extensive settlement-caused brickwork damage mars a downtown building in Montrose.**

Figure 3-7. ***C*****, Settlement has damaged the porch and doorway of a home in Glenwood Springs.**

Figure 3-7. ***D*****, Pseudokarst collapse from dispersion and piping erosion has caused this agricultural field to be abandoned near Olathe. The plateau in the background is Grand Mesa.**

Figure 3-7. ***E*****, The wood flooring in a school in Montrose has buckled because of compression due to settlement of the underlying concrete slab-on-grade. Photo courtesy of Tom Griepentrog, Buckhorn Geotech.**

Figure 3-7. ***F*** **and G, Large wetting events such as broken water mains will cause widespread settlements, ultimately putting the soil of the surrounding area into tension (which is why arcuate cracks form around wetting ponds). In these two photographs, soil is being pulled down and away from the foundation walls. Note the gap along the foundation wall and the distress of the water line into the home. Dangerous conditions occur if natural gas or buried power lines are pulled to the point of rupture.**

H **and** ***I*****, Large-scale settlement due to a water main break in road (off picture) has pulled this driveway down and away from the residence. This residence was previously underpinned, and a new leveling concrete pad was poured in the garage. The pipe piles were of inadequate depth, and deeper wetting of the collapsible soils caused further distress to the structure. (In** ***I*****, a baseball hat shows the scale).**

4. PROPERTIES OF COLLAPSIBLE SOILS

This chapter describes some of the basic engineering index properties of the various types of collapsible soils and some of the soil-test criteria that are used to determine the potential and severity of collapse. Geotechnical engineers and soil scientists use standard tests to classify soils; this chapter also presents the range of test results that determine when soils might be susceptible to collapse.

HYDROCOMPACTIVE SOILS

This subsection explains the engineering index properties, diagnostic analytical tests, and empirical methods (based on certain soil-test results) that have been developed to determine collapse susceptibility of hydrocompactive soils. The information includes data collected for the Colorado Statewide Collapsible Soil Study.

Soil Classification—Gradation and Plasticity

For geotechnical engineering purposes, the Unified Soil Classification System (USCS) is most commonly used. The basic reference used in this publication is the American Society of Testing and Materials (ASTM) D 2487-06, *Standard Classification of Soils for Engineering Purposes*. Soils are classified on the basis of gradation (i.e., the percentage breakdown of the size fractions of soil constituents) and the plasticity behavior (i.e., how easily the soil can be molded) of the fine-grained fraction of the soil sample through a range of water contents.

The gradation test involves the mechanical sorting, by vibration or shaking, of soil using a set of sieves, which are stackable circular trays with differing mesh size openings. The soil sample is weighed and then sieved through the stack; the part of the soil sample retained in each sieve is then weighed, and the grain-size percentage is calculated. A grain-size distribution curve is then created to graphically illustrate the test results.

Once the gradation is complete, plasticity tests, known as the Atterberg limits, are performed on the fine-grained (silt and clay) part of the soil. Two boundaries are determined. The liquid limit (LL) is the water content (as a percentage) when the soil begins to "flow"; the LL defines the boundary between plastic and viscous fluid states. The plastic limit (PL) is the water content (as a percentage) at which the soil stops behaving plastically and starts to crumble. These tests are standardized by ASTM in publication D 4318-00, *Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils*. The plasticity index (PI) is measured as the difference between the LL and PL and generally defines the ability of a clay soil to retain moisture. High LL and PI values indicate "fat clays" or swelling soils that can hold high percentages of water, generally with a corresponding increase in volume and swell pressure. Lower LL and PI values indicate less plastic "lean clays" that have a lesser ability to retain moisture or a soil in which silt is the primary constituent. Most collapsible soils fall into the lower PI category.

Figure 4-1 is a simplified chart that explains how to classify soils by using the USCS. The key points are as follows:

1. The no. 200 sieve used in soil gradation retains all gravel and sand and only passes the silt and clay parts of the soil sample; the silt and clay particles are <0.075 mm in size, or smaller than the period at the end of this sentence. It is important to emphasize that the size fraction that passes the no. 200 sieve plays an important role in both collapse and swell characteristics of a soil. Although collapse is common in soils with high clay and silt concentrations, collapse can also occur in soils with as little as 15 percent fine material that passes the no. 200 sieve.

2. The classification of the fine-grained part of the soil that passes a no. 200 sieve (i.e., the clay and silt) is based on the plasticity chart.

3. For the purpose of this study of collapsible soils, organic soils are not considered and have been removed from figure 4-1. Though organic soils usually have significant settlement properties, they do not fall into the definition of collapsible soils used here or in widely published references.

4. Swelling soils have the ability to take in large volumes of water when they swell; hence they have high LL values (greater than 50) and high PI values (generally from approximately 20 to 40 or higher). Referred to as "fat clays," they are classified into the CH and MH groups.

5. Collapsible soils generally have lower LL and very low PI; they are mostly classified into the CL, ML, and CL-ML groups. Coarser collapsible soils (i.e., gravel and sand with a fine-grained collapsible matrix) can be classified as GM, GM-GC, GC, SM, SM-SC, or SC.

Major divisions			Group symbol	Group name
Coarse-grained soils more than 50% is retained on No. 200 sieve	gravel >50 percent of coarse fraction retained on No. 4 sieve	clean gravel	**GW**	well-graded gravel, fine to coarse gravel
			GP	poorly graded gravel
		gravel with >12 percent fines	**GM**	silty gravel
			GC	clayey gravel
	sand ≥50 percent of coarse fraction passes No. 4 sieve	clean sand	**SW**	well-graded sand, fine to coarse sand
			SP	poorly graded sand
		sand with >12 percent fines	**SM**	silty sand
			SC	clayey sand
Fine-grained soils more than 50% passes No. 200 sieve	silt and clay liquid limit is ≤50	inorganic	**ML**	silt
			CL	clay
	silt and clay liquid limit is ≥50	inorganic	**MH**	silt of high plasticity, elastic silt
			CH	clay of high plasticity, fat clay

1. Modifiers are used for specific gradation percentages to subdivide each group.
   Example:
   Clay with sand—more than 50 percent clay with 15 to 29 percent sand
   Sandy clay—more than 50 percent clay with greater than 30 percent sand
   Sandy clay with gravel—more than 50 percent clay, 30 percent or more sand, and the remainder gravel—but not less than 15 percent.
2. Dual symbols are used for sand and gravel with 5 to 12 percent fines.
   Example:
   GW–GM indicates well-graded gravel with silt

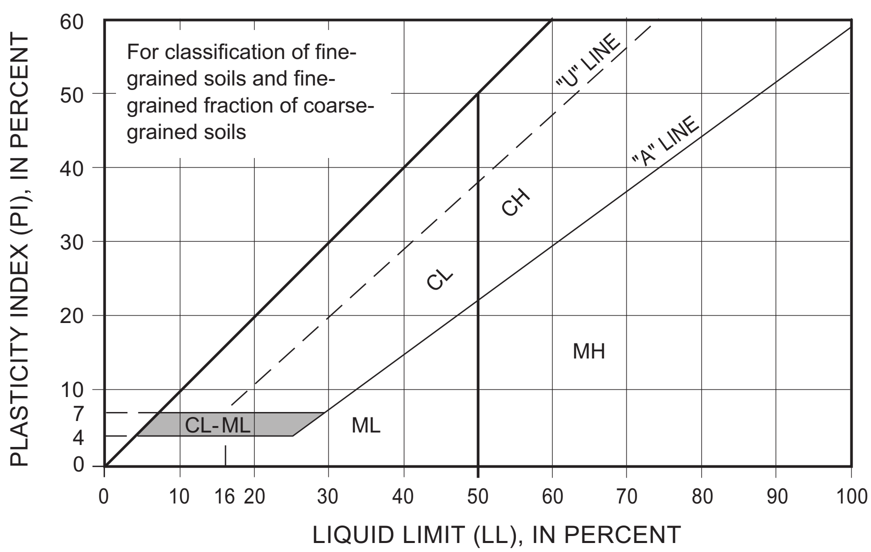


Figure 4-1. The United Soil Classification System (simplified). Organic soils classifications have been removed from this simplified chart. The "A" line in the lower graph marks the separation of inorganic silt and clay based on the relationship of PI to the LL. The "U" line is the upper limit of the relationship of the PI to the LL in fine-grained soils.

Dry Density

Dry density (DD) is another important index property for soils that can indicate a susceptibility to hydrocompaction. By definition, dry density is the density of a soil volume that has been oven dried to remove all pore water. Dry density is expressed in units of pounds per cubic foot (pcf) and metrically as grams per cubic centimeter (g/cm^3), as used in Natural Resources Conservation Service (NRCS) soil surveys. A soil sample of a known volume is weighed, oven dried, and then reweighed. The weight of the oven-dried soil is then divided by the original volume and expressed in the unit weight. This test is standardized by ASTM as D 4254-00, *Standard Test Methods for Minimum Index Density and Unit Weight of Soils and Calculation of Relative Density*. **Table 4-1** shows some general values of different soil and rock types.

Table 4-1. Densities of Representative Rock and Soils.

MATERIAL	DENSITY (pcf)
Granite (solid)	165
Moist sand and gravel with clay or silt matrix	125
Front Range high-plasticity clay soil	110–120
Highly collapsible, dry, nonplastic, clayey silt	70–95

Moisture Content

The moisture content (MC) is the percentage weight of pore water in a soil sample. It is a ratio, expressed as a percentage, that is calculated by dividing the weight of water in a soil sample by the weight of the solids in the soil sample. The weight of the water is the difference between the measured weight of the soil sample in its natural state and the weight after it has been oven dried (i.e., the weight of the solids). The standardized ASTM test is D 2216-05, *Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass*. The MC of dry soils generally ranges from 3 to 10 percent. By comparison, greater than 12 percent is the typical MC of fat, high-plasticity clay soils and saturated soils or soils within a ground-water table.

Methods to Determine Collapse Susceptibility

Collapse potential (CP) is typically determined by a laboratory testing method that is also used to determine swelling potential of expansive soils. In addition, in situ tests and empirical methods use certain index properties to quantify and qualify CP.

Although several tests are described in this section, the consolidometer testing of soil samples in the laboratory and the field soil-saturation plate load tests will generally provide the most accurate and site-specific determination of CP and related hazard potential, provided the precautions are considered.

Swell/Consolidation Soil-Test (Oedometer or Consolidometer Test

Soil 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, with their in situ moisture content, are hydraulically jacked into a confining ring, trimmed, precisely measured, and inserted into a test chamber with porous inserts or stones at the top and bottom. A dial gauge is placed on the top of the soil sample container and zeroed (**figure 4-2**). The sample is then incrementally loaded with weights to a specific surcharge (that is, the load representative of the weight of the soil or a residential foundation) pressure. After this, the soil is flooded and allowed to saturate; the percent collapse or swell recorded at that constant pressure is the ratio of the change in height or thickness of the soil after wetting to the original sample height. Collapse potential, expressed as a percentage, is defined as

$$CP = (\Delta H_c / H_0) \times 100,$$

where ΔH_c is the change in height, measured by the dial gauge, of the sample after saturation and H_0 is the initial height of the original sample at its in situ moisture content. The soil is then further incrementally loaded to determine the compression curve.

During surcharge loading, collapsible soils usually compress only slightly, but when the sample is flooded at constant stress (or load), such soils undergo considerable collapse, which results in a reduction in the volume of the sample. An example of a consolidation chart (**figure 4-3**) shows an initial 2 percent collapse of a sandy-silty clay soil sample, followed by significant additional collapse (7 percent) upon wetting, and further consolidation upon additional loading. The shape of the postwetting curve on a consolidation chart depends on the soil composition. For example, if a soil has a high clay content, further consolidation with additional loading can produce a very steep curve. In a more granular hydrocompactive soil, however, the collapse is essentially complete after wetting, and the continuation of the curve on the consolidation chart is flattened (i.e., less steep). A representation of such a flattened curve is also shown in figure 4-3.

The severity criteria for soil collapse, based on the percentage decrease of a sample height before and after saturation, were developed by Jennings and Knight (1975) and are shown in **table 4-2**. The criteria are based on a wetting load of 4,200 pounds per square foot (psf) or 2 tons per square foot (tsf), with subsequent flooding of the soil sample for 24 hours. This wetting load is high and not generally representative of normal residential loads for wood-frame construction.

Figure 4-2. Four test apparatuses for swell/consolidation soil testing. To load soil sample, weights are hung through arm. The soil sample is hidden in a steel confining ring shown with lettering in the insert photo. Note that the confining ring within the transparent cylinder is flooded with water. Insert photo also shows dial gauge that measures the swell or collapse displacement of the soil sample by deflection of the lever arm when it is wetted. After flooding and a prescribed period of consolidation, the sample is incrementally loaded by additional weights added from the shelf below. Lab access courtesy of Yeh and Associates.

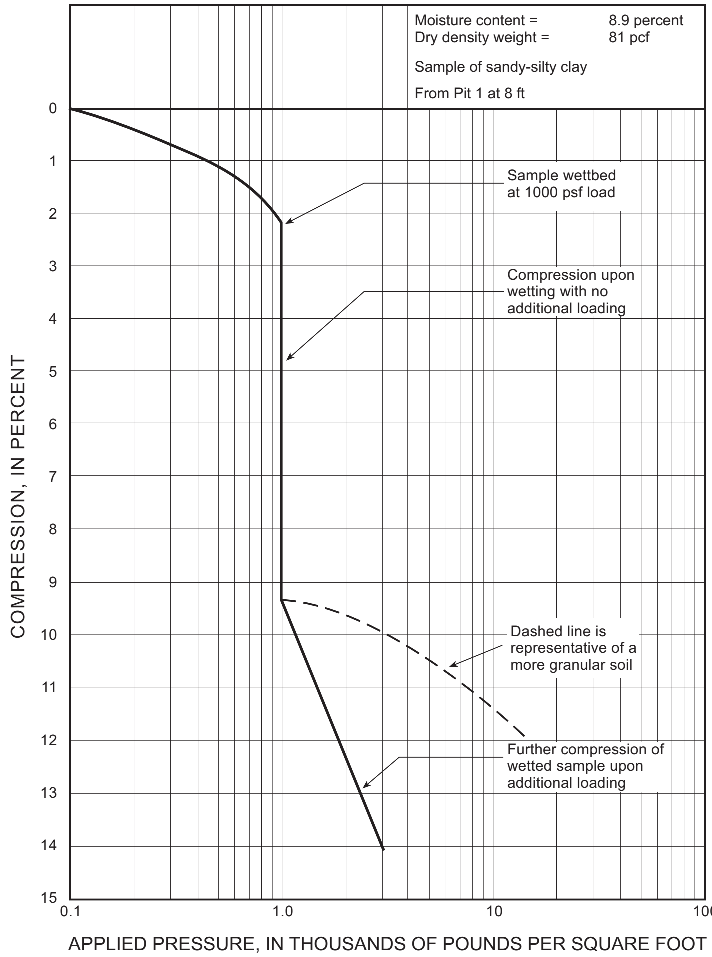


Figure 4-3. Swell/consolidation chart from a geotechnical consultant's report showing a Colorado test sample of low-density (81 pcf) sandy-silty clay soil that collapsed more than 7 percent after wetting at a constant surcharge load of 1 ksf (= 1,000 pounds per square foot). The dashed line is a typical consolidation curve of a more granular soil after the initial wetting.

Table 4-2. Severity Criteria for Soil Collapse with test load of 4,200 psf (Jennings and Knight, 1975).

COLLAPSE POTENTIAL (%)	Severity of Problem
0–1%	No problem
1–5%	Moderate problem
5–10%	Trouble
10–20%	Severe trouble
>20%	Very severe trouble

Mock and Pawlak (1983) established the criteria shown in **table 4-3** for collapse hazard potential based on their own experience in west-central Colorado. Their model follows the one proposed by Jennings and Knight (1975), but uses a reduced surcharge of 1,000 psf when the test sample is saturated. This load, similar to that used for swelling soil hazard assessments, has become the standard most commonly seen in central, and Front Range, Colorado.

In a double consolidometer test, in addition to one sample being wetted, an "identical" sample at its natural moisture content is also loaded simultaneously but not saturated. The natural and wet consolidation curves are plotted together to compare the natural and wet conditions of the soil sample under the same load.

Table 4-3. Criteria for Collapse Potential with test load at 1,000psf (Mock and Pawlak, 1983).

COLLAPSE POTENTIAL (%)	HAZARD
0–1%	No problem
1–3%	Low
3–5%	Moderate
>5%	High

Generally, a postconsolidation dry density is measured afterward to determine the increase in density (loss of void space) after wetting and loading.

Consolidometer collapse testing on "undisturbed" soil samples under light loading and flooded conditions is the most common method to determine the collapse potential of soils. To sample a soil at its in situ moisture content, drilling fluids cannot be used; test borings are generally drilled by auger or various hammer drill and rotary tri-cone methods using compressed air to remove cuttings. The geotechnical engineering community places strong emphasis on the results of this test for foundation design. However, there are limitations of swell/consolidation tests, as described next.

One concern is the quality of the "undisturbed" soil sample that is collected and whether the swell/consolidation test results truly represent the in situ conditions of the soil. Although it is possible to collect subsurface samples by methods that produce very little disturbance (e.g., by continuous sampling in hollow-flight augers or by collecting a block of soil from an excavation or test pit and then shaping the block to insert into a test ring), most samples are collected by driving a thin-walled metal tube into the soil (a California tube or sampler is most common) and then hydraulically extruding this sample from the tube into the test ring. Driving a 2-in. outside-diameter brass tube 12 in. into a soil with a drill-rig hammer can disturb the soil being sampled by either compressing or loosening it, and many geotechnical experts question whether such samples can be called even "relatively undisturbed." Some engineers with experience in collapsible soils prefer the 3-in. sample with brass ring liners (Hansen and others, 1989). The sample does not need to be extruded and potentially further disturbed. The liner rings with sample can be placed directly into the consolidometer apparatus. Even with this technique, some disagreement exists in the literature about the level of sample disturbance and how it affects consolidation results. Jennings and Knight (1957) mentioned as much as 30 percent compression of certain types of looser, low-density soil during sampling by thin-wall sampling tubes, which would densify the soil and skew the consolidation test results to lower CP values. Conversely, Dudley (1970) noted that in some situations, calculated settlements would be as much as twice the actual settlement. In support of this conclusion, Day (1990) found that the driving disturbance by a falling hammer tended to fracture the sample. These fractures increase the volume and cause higher CP values in samples, compared to testing of shaped-block samples of undisturbed soil. However, Houston and El-Ehwany (1991) stated the opposite, that there is little difference in the quality of sampling between thin-walled samplers driven into cemented collapsible soil and shaping block samples to fit the consolidation ring. Our experience is that driving a sampler into very hard, very dry hydrocompactive soil can result in fracture and breakage of the soil sample, which could skew the swell/consolidation test results to CP measurements that are higher than those of in situ subsurface soil. Commonly, these types of soil samples tend to break into thin cross-sectional disks as the drill-rig hammer drives the sampler tube into the soil. A somewhat moister, less cemented, or homogeneous soil sample, such as windblown loess, might not behave the same way and could compact further when tube sampled, as reported by Jennings and Knight (1957) and by Rollins and others (1992b) in their discussion of the Houston and El-Ehwany (1991) work.

Another concern is the lack of assurance that the particular soil sample is representative of the subsurface soil at that location. Client budgetary concerns can result in fewer test borings and (or) less sampling and testing. In Colorado, almost all subsurface investigations by drill rig for residential foundation design are performed with a split spoon or California spoon and sampled every 5 or 10 ft, or only once for the entire boring within what is assumed to be the stress-distribution zone of a shallow foundation. Although this sampling interval might be acceptable where the soil is consistent, such as in a thick mantle of loess, most collapsible-soil terrain in Colorado originated in depositional systems that can have significant lateral and depth variability, and corresponding differences in soil properties. In many cases, including test pits in addition to borings as part of the subsurface soil investigation could be beneficial, as the near-surface soil column could be observed. Sampling intervals, by either block or manual tube sampling, can then be selected at soil horizons or strata that would best represent the overall properties of the subsurface soil.

In gravelly and rocky collapsible soils, collecting an acceptable sample in a thin-walled metal tube is next to impossible. In such circumstances, only samples from the finer-grained part or layer of a soil deposit can be taken for testing, and the potential for error, as already mentioned, would also be present.

In Situ Testing Options

Other methods to determine the potential for soil collapse are in situ tests completed during preliminary investigations prior to design and development. Most in situ tests artificially introduce moisture to the soil, and the subsequent behavior is observed and measured. The easiest method is to create a pond by excavating a shallow depression and filling it with water. By installing monuments or other instrumentation, rates and magnitudes of settlement can be measured. Ponding also allows a person to observe the physical manifestations of soil collapse and ground settlement (e.g., sags, arcuate soil cracks, piping holes, etc.) (**figure 4-4; also see fig. 3-4**). However, ponding or surface wetting is somewhat unreliable to determine whether a complete, uniform wetting of the subsoils is occurring. Case histories have shown that a saturation front is rarely uniform. Forensic investigations of damaged structures in Colorado have shown fully saturated soils, which have settled, surrounding bone-dry, uncollapsed soils at, or very near, their natural low-moisture content. Ed Church, personal commun., 1998). El-Ehwany and Houston (1990), in their work on settlement and moisture movement in collapsible soils,

Figure 4-4. Ground settlement at test ponds along the proposed Interstate 70 alignment west of Rifle during preliminary studies in the mid-1970s. Site is on a large alluvial fan on the north valley side of the Colorado River. Note arcuate cracks and subsidence of test embankment. During the test, the ponded water sometimes completely drained into a network of pipes formed in the subsurface soil. Photos courtesy of R. Barrett and CDOT.

discussed similar observations of relatively sharp wetting fronts that they attributed to soil suction (which dominates over gravity as a control of water flow) and soil type. They recommended procedures to accurately determine depth and lateral extent of water migration during field ponding tests.

A field plate load test is another analytical method. This expensive and time-consuming test is rarely done for residential development but is commonly used in highway and canal alignment studies. Compared to a laboratory-based consolid- ometer test, a plate load test more effectively measures the actual, in situ field behavior of collapsible soils under load, excludes many of the uncontrollable factors (e.g., soil disturbance, inability to recover a sample, soil heterogeneity, etc.), and minimizes the effects of the "scale" factor (Reznik, 1992, 1993). There have been studies to correlate plate load and consolidometer collapse testing, mostly in fairly uniform soils such as loess (Reznik, 1995).

In plate load tests, topsoil is removed, and a skid or pad is placed on the native subsoil. Weights, as simple as sandbags or concrete blocks, are placed on the pad to a predetermined pressure uniformly bearing on the soil. An initial ground survey is performed, and the ground and subsurface are then wetted. The soil can be wetted by ponding water within berms around the load pad or introducing water via borings that are drilled through the column of soil that is presumed to have the potential to collapse. The drill boring is cased with slotted pipe and serves as an infiltration well through which water is pumped or allowed to free-flow into the subsurface. Kyle M. Rollins at Brigham Young University very effectively illustrated the value of plate load tests in his work to evaluate six types of mitigation in hydrocompactive soils in Nephi, Utah (Rollins and Rogers, 1994).

In practice in the State of Colorado, plate load testing to determine collapse potential has been done on loess soils in the Pueblo area (Lofgren, 1969), for irrigation canal construction by the U.S. Bureau of Reclamation (USBR) near Cortez (**figure 4-5**) (Luehring, 1988), and for the Highway 82 Snowmass Canyon Project by the Colorado Department of Transportation (CDOT) (CDOT and others, 2000).

Figure 4-5. An inexpensive but effective load test by the USBR near Mesa Verde, south of Cortez. *A*, Sand bags of known weight are placed on a bearing plate above a well boring. *B*, Water from a plastic tank is allowed to infiltrate outward from the well at a constant rate. Photos courtesy of R. Luehring.

Research has also been done on down-hole collapse tests whereby a load apparatus is staked over an open boring, a steel pipe with a small circular load plate is placed at the bottom of the borehole, and a load platform is placed on top at the surface. A water port is used to flood the boring through the steel pipe. The load platform has a dial measurement gauge so that deflection of the loaded pipe can be measured after water has been introduced. More information on this test method can be found in Bowers (1986) and Houston and others (1995). It is not known whether this method has been used in Colorado.

There is another simple field test known as a "sausage" field test that requires no complicated sampling or expensive in situ test setups (Jennings and Knight, 1975). A block of material is sampled and broken into two pieces about hand sized. Each is trimmed until the observer considers they are of equal volume. One is then placed in a plastic bag and wetted and molded or packed by hand to form a damp ball. The volume of this ball is then compared with the volume of the undisturbed piece. If smaller, then collapse is suspected. This field test is quite crude relative to the other tests described and should not be used as a stand-alone determination of collapse potential.

Empirical Methods to Determine Collapse Susceptibility

There are several empirical methods to determine collapse susceptibility. Although some are generally derived from the common soil-index tests, many are rather complex and require special soil testing and a fundamental understanding of soil mechanics (e.g., specific gravity, void ratio, porosity, degree of saturation, shear, and strain). Some methods have been developed into susceptibility charts that graph a relationship between different soil-index properties or into equations that give collapse coefficients. Certain methods have been developed as site-specific regression equations (Reimers, 1986), whereas others are as simple as single indicators. For example, Feda (1966) mentioned a simple collapse indicator—porosity; if natural porosity is more than 40 percent, the soil should be considered collapse susceptible. Many of the methods were developed for blanket deposits of loess, which have relatively uniform soil properties. These methods are generally less reliable for highly variable alluvial-fan and colluvial soil deposits (Beckwith and Hansen, 1989) or for the more plastic swelling-clay soils. It is beyond the scope of this publication to discuss, in depth, the various soil-mechanics methods that have been proposed over the years. The reader is encouraged to review Reimers (1986), Luehring (1988), Huang (1989), and Roullier and Stilley (1993), which contain informative discussions on the various analytical, regression-based, and graphical methods.

Three different collapse-susceptibility charts have been reviewed for this publication: (1) the relationship of dry density (DD) and liquid limit (LL), proposed by Gibbs and Bara (1962) (**figure 4-6**); (2) the relationship of DD and percentage of soil passing the no. 200 sieve, presented by Mock and Pawlak (1983) (**figure 4-7**); and (3) the comparison of DD and moisture content (MC), proposed by Roullier and Stilley (1993) and Karakouzian and Roullier (1993) (**figure 4-8**). All three methods empirically compare collapse-potential test results to common soil-index property tests that are the least expensive and most commonly performed in the normal course of a geotechnical investigation. DD is inversely proportional to porosity and void space (i.e., the lower the density, the higher the void space and porosity); MC is a reflection of the degree of saturation of a soil sample; the percentage of fines passing the no. 200 sieve is a measure of the clay and silt content in the sample; and the Atterberg limits—LL and PI—provide an indication of the composition of the clay and silt.

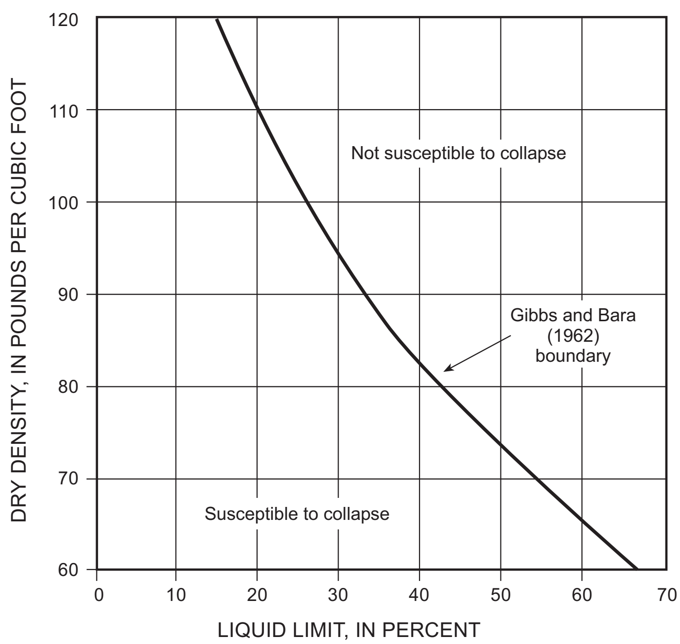


Figure 4-6. The relationship of dry density (DD) versus liquid limit (LL) for collapsible soils, showing the Gibbs and Bara (1962) susceptibility boundary.

Colorado Data Evaluated by Collapse-Susceptibility Charts

Part of the Colorado study included the collection of case histories throughout Colorado where collapsible soils have been found. In addition to making interpretations of the surficial geologic conditions, sediment deposition, and geomorphology, this study also included the compilation of soil-test data in which collapse potential (CP) was measured from modified

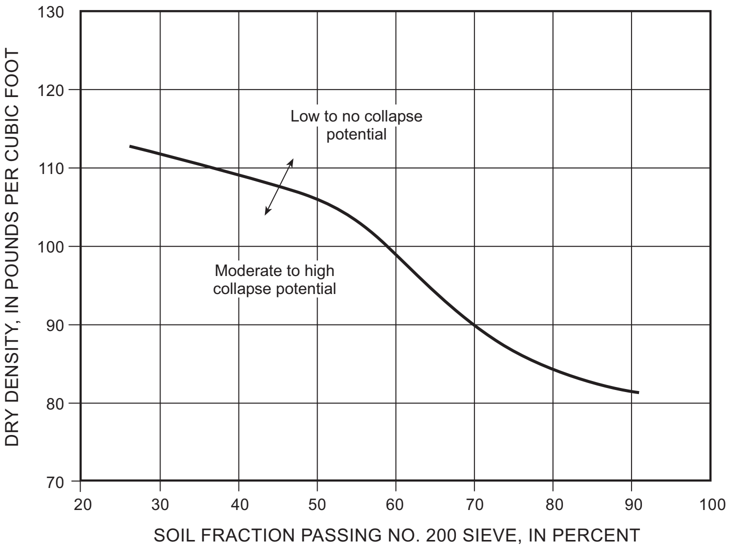


Figure 4-7. The relationship of dry density (DD) versus percentage of soil passing no. 200 sieve for collapsible soils, showing the Mock and Pawlak (1983) susceptibility boundary.

swell/consolidation tests performed by geotechnical consultants working in Colorado. The tests were originally conducted as part of investigations for residential development or as part of forensic investigations where damage due to soil collapse and settlement had occurred. The majority of the soil sampling was in the 3- to 6-ft depth range, where shallow footings would be placed below the frost line. The deepest sample tested in our database was collected at 40 ft. Most of the consolidation data had associated MC and DD measurements. Fewer data points also had gradation measurements, showing the percentage of clay and silt, and LL and PI measurements.

In addition to the statewide data on collapsible soils, data sets from two other projects were used: (1) the soil-test data compiled by Berry and others (2002) from the far southern Denver metro area of Highlands Ranch, which has a complex mix of alluvial, colluvial, and eolian sediments and where the soils are both expansive and hydrocompactive; and (2) for comparison purposes, a data set compiled by Noe (2005) from predominantly expansive clay soils of the Roxborough area of Jefferson County in the far southwestern Denver metro area, within the steeply dipping bedrock zone. Only test results for soils were used; bedrock samples from the Douglas County and Roxborough studies were excluded.

The collapse-potential (CP) data collected for this comparative study were generally compiled from consolidometer tests where the samples were wetted at a load of 1,000 psf. For certain tests, predominantly from the western slope, some samples were wetted at lower surcharge loads. In those cases, the percentage

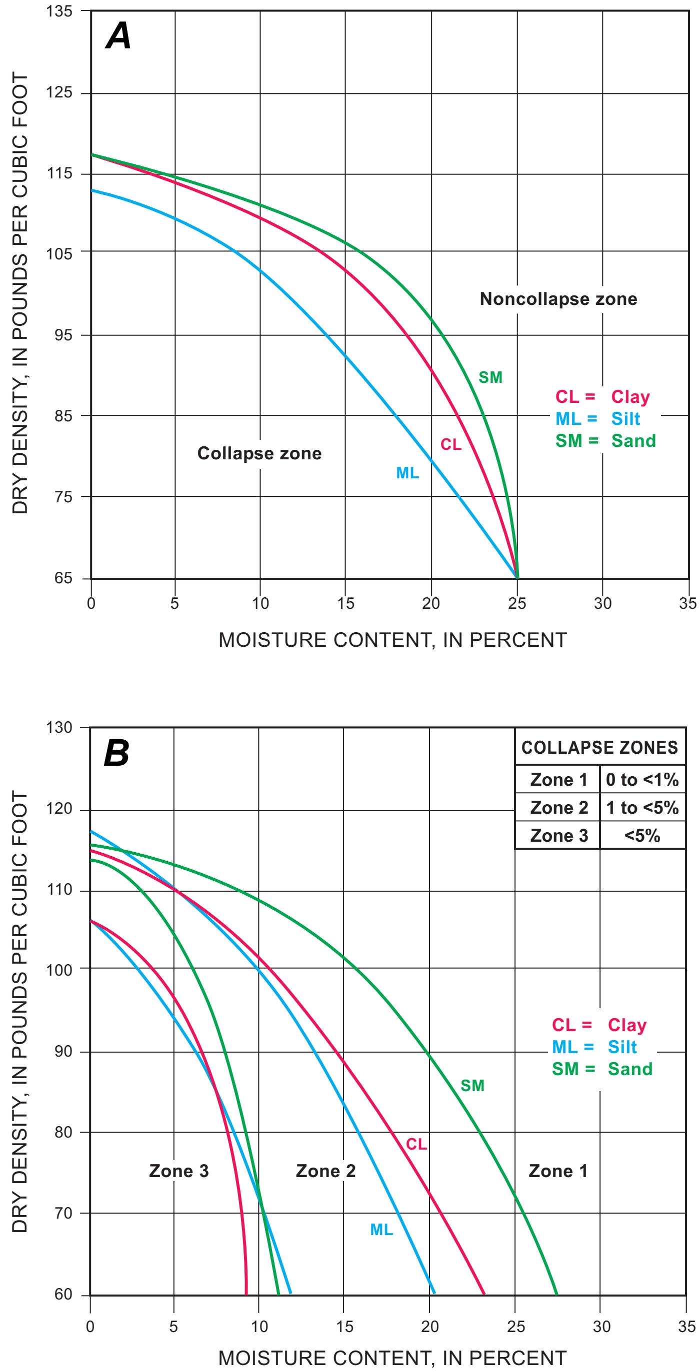


Figure 4-8. The relationship of dry density (DD) versus moisture content (MC) for collapsible soils, showing collapse-susceptibility boundaries. *A*, Modified from Roullier and Stilley (1993). *B*, Modified from Karakouzian and Roullier (1993), who proposed three zones based on severity of collapse.

of collapse was interpolated from the consolidation curve at the intercept with the 1,000-psf load line. These data were plotted by using the three susceptibility graphical methods described in the previous section. CP values were grouped on the basis of the collapse-potential chart from Mock and Pawlak (1983) that was shown in table 4-3.

Dry Density Versus Sample Percent Passing No. 200 Sieve

In **figure 4-9**, CP data points were plotted on a chart of DD versus percentage of soil passing the no. 200 sieve. The data points of test data where gradations were available for both the statewide (fig. 4-9*A*) and Douglas County data sets (fig. 4-9*B*) show reasonable correlation with the designated collapse-potential fields on each side of the Mock and Pawlak (1983) boundary line on the chart, except for the statewide data that have higher percentages of fines. For the Douglas County data set, data points showing soil swell generally plot within the low- to no-collapse field. The pattern of the statewide collapse data suggests that low dry density is the major factor in collapse potential. At a density of about 105 pcf, a near-horizontal line can be drawn that bounds the area containing nearly all samples, independent of the soil sample percentage that passed the no. 200 sieve.

Dry Density (DD) Versus Liquid Limit (LL)

Data-set plots of CP from test data where both DD and LL values were measured are shown in **figure 4-10**. High-CP soils and the lower-LL soils from the Colorado statewide data set show some correlation by plotting on the "susceptible to collapse" side of the Gibbs and Bara (1962) boundary line (fig. 4-10A). The data set plotted in figure 4-10A approximates similar groupings shown in Owens and Rollins (1990) and Rollins and others (1992a) for comparable soils in Utah. Although those test samples with high CP values fall easily below the boundary line, other samples with slight to moderate CP values plotted within the stable side of the graph. The correlation is very weak in the data set of clay-rich, eolian–source collapsible soils of Douglas County, which have higher liquid limits and

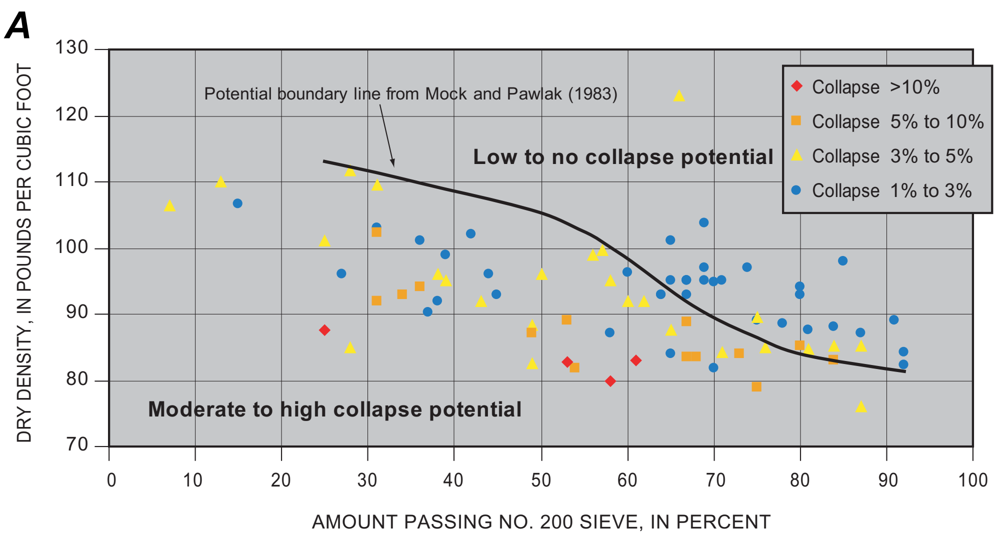
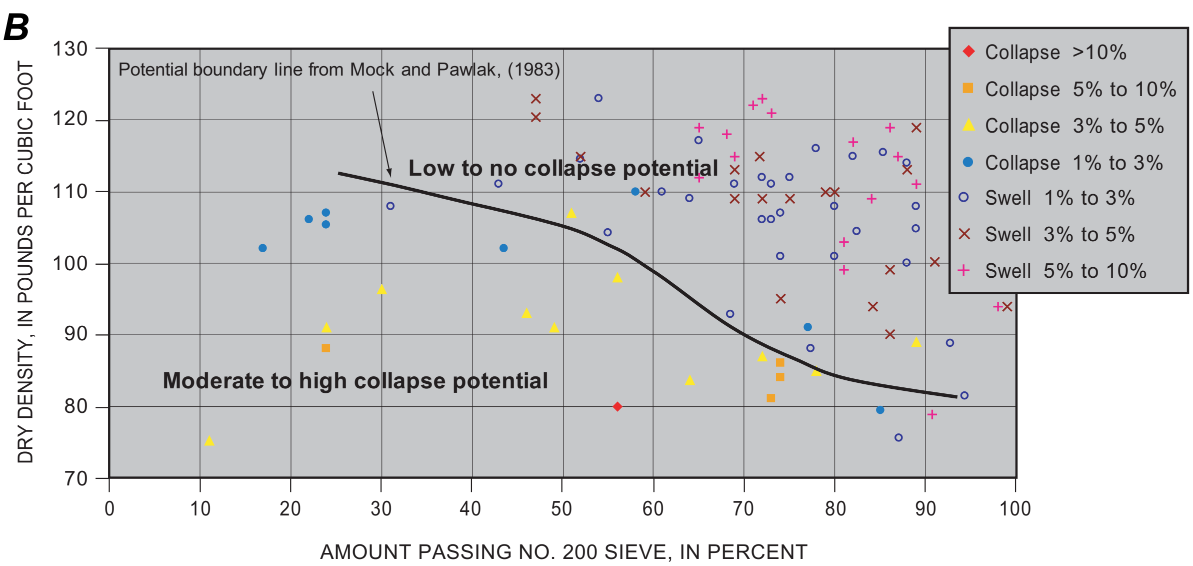


Figure 4-9. Soil collapse-potential test data and the Mock and Pawlak (1983) collapse-susceptibility boundary. Data are from swell/consolidation test results when the soil is loaded to 1,000 psf and then wetted.
***A*, Statewide case-history soil-test data.**
***B*, Douglas County soil-test data from Berry and others (2002).**



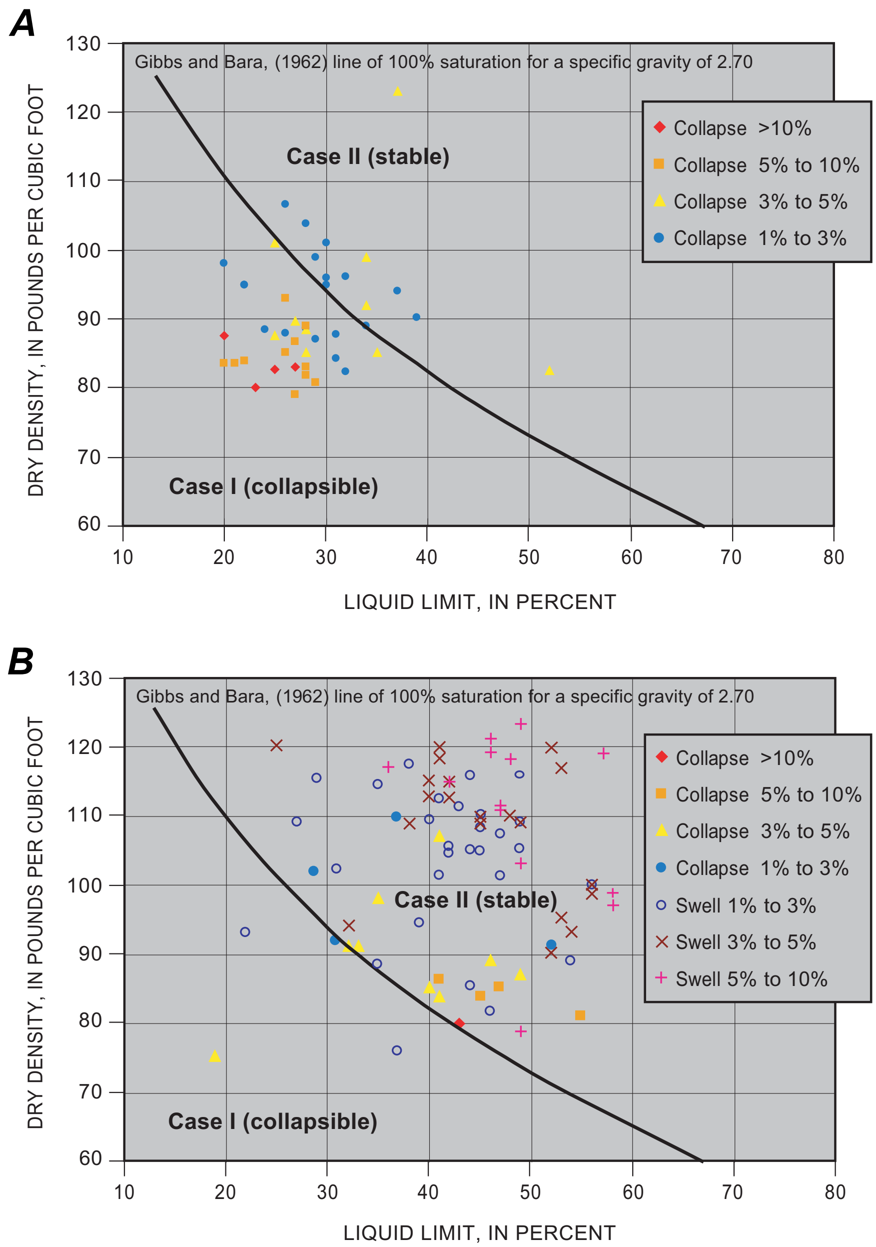


Figure 4-10. Soil collapse-potential test data with Gibbs and Bara (1962) collapse-susceptibility boundary. Data are from swell/consolidation test results when the soil is loaded to 1,000 psf and then wetted.

***A*, Statewide case-history soil data.**

***B*, Douglas County soil data from Berry and others (2002).**

reflect the presence of swelling clays (fig. 4-10B). For that reason this method of determining hydrocompaction susceptibility should not be considered reliable, especially in clay-rich collapsible soils, an assessment shared by Prokopovich (1984).

Dry Density (DD) Versus Moisture Content (MC)

The relationship of dry density and moisture content provides the best indication of collapse potential for Colorado. Through the use of data from the Douglas County study, the Roxborough study, and the statewide study where test points showed either collapse or expansion that exceeded 1 percent, two distinct plot populations become apparent (**figure 4-11A**).
With further restrictions of the test range to soil collapse or expansion greater than 3 percent, these plot populations become increasingly distinct; **figure 4-11B** clearly shows the differences in dry density and moisture content between moderately and highly collapsible and expansive soils in Colorado. The boundary between these two soil populations is similar to the collapse boundaries discussed in Roullier and Stilley (1993) and Karakouzian and Roullier (1993) (fig. 4-8), which were based on collapsible-soil data from the Las Vegas Valley in Nevada. In **figure 4-12**, graphs of only Colorado collapse data, superimposed on the Karakouzian and Roullier (1993) and Roullier and Stilley (1993) boundary lines, reveal that very few samples with significant CP exceed an MC of 15 percent. Generally, for collapsible soils in semiarid Colorado, the instances in which moisture contents exceed 15 percent are

A

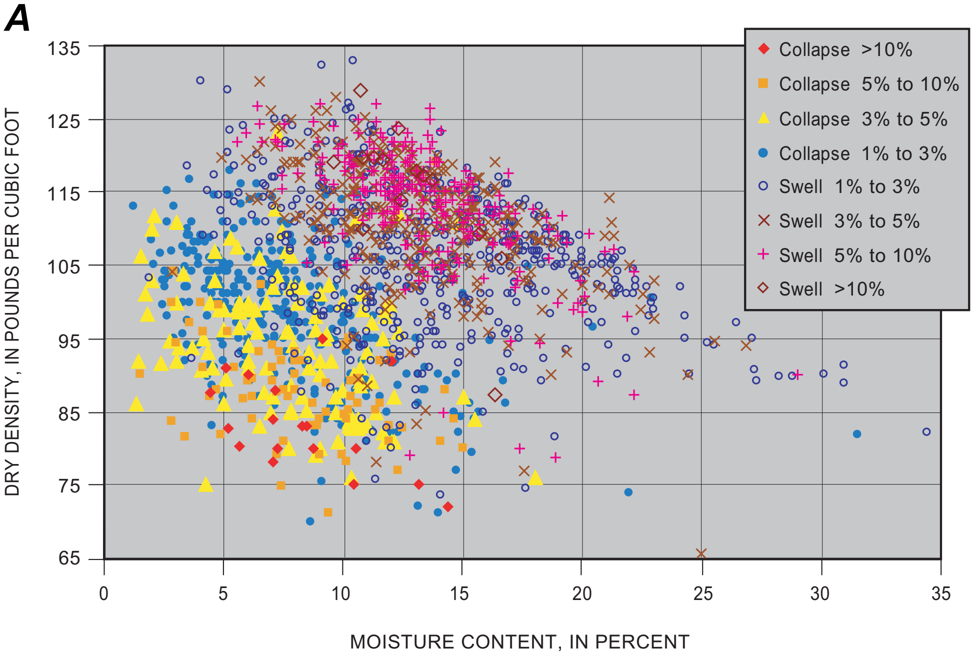
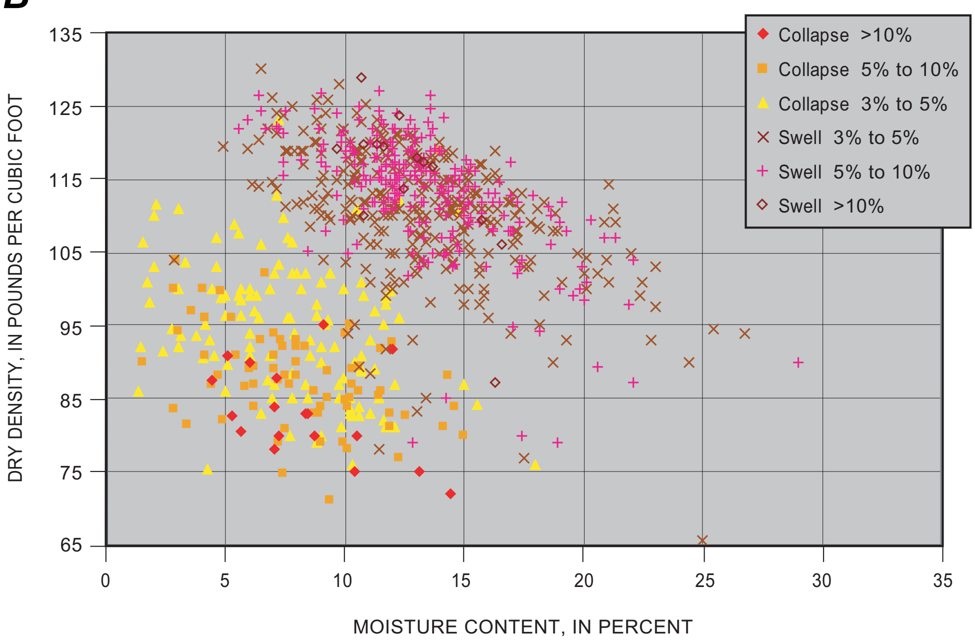


Figure 4-11. The relationship of dry density versus moisture content in the statewide soil-collapse data, including data from the Douglas County and Roxborough studies. Data are from swell/consolidation test results when the soil is loaded to 1,000 psf and then wetted.

***A*, Swell/consolidation plots of greater than 1 percent swell and 1 percent collapse.**

***B*, Swell/consolidation plots of greater than 3 percent swell and 3 percent collapse.**

B



usually reported from forensic investigations where the subsurface soils already have elevated moisture, the saturation threshold has been exceeded, and collapse and settlement have begun.

There are two areas where Colorado data depart from the Roullier and Stilley (1993) graph of dry density versus moisture content. Roullier and Stilley further differentiated the collapsible and noncollapsible zones by three soil classifications; silt (ML) showed the highest collapse, followed by clay (CL) and sand (SM) (fig. 4-8*A*.) This categorization of collapse potential by soil classification could not be discerned for Colorado samples. Also, although there is generally very good correlation at the lower moisture contents, the Roullier and Stilley (1993) boundary lines define a collapse zone with moisture contents that are too high (i.e., greater than 15 percent) to be considered having collapse potential in Colorado (fig. 4-12*A*).

Karakouzian and Roullier (1993) refined the collapse versus noncollapse division based on the sand, silt, and clay soil classification, with paired boundary lines that differentiate three different zones of collapse probabilities (fig. 4-8B). The chart in figure 4-12B shows the relationship of these proposed collapse zones to the grouping of the Colorado data

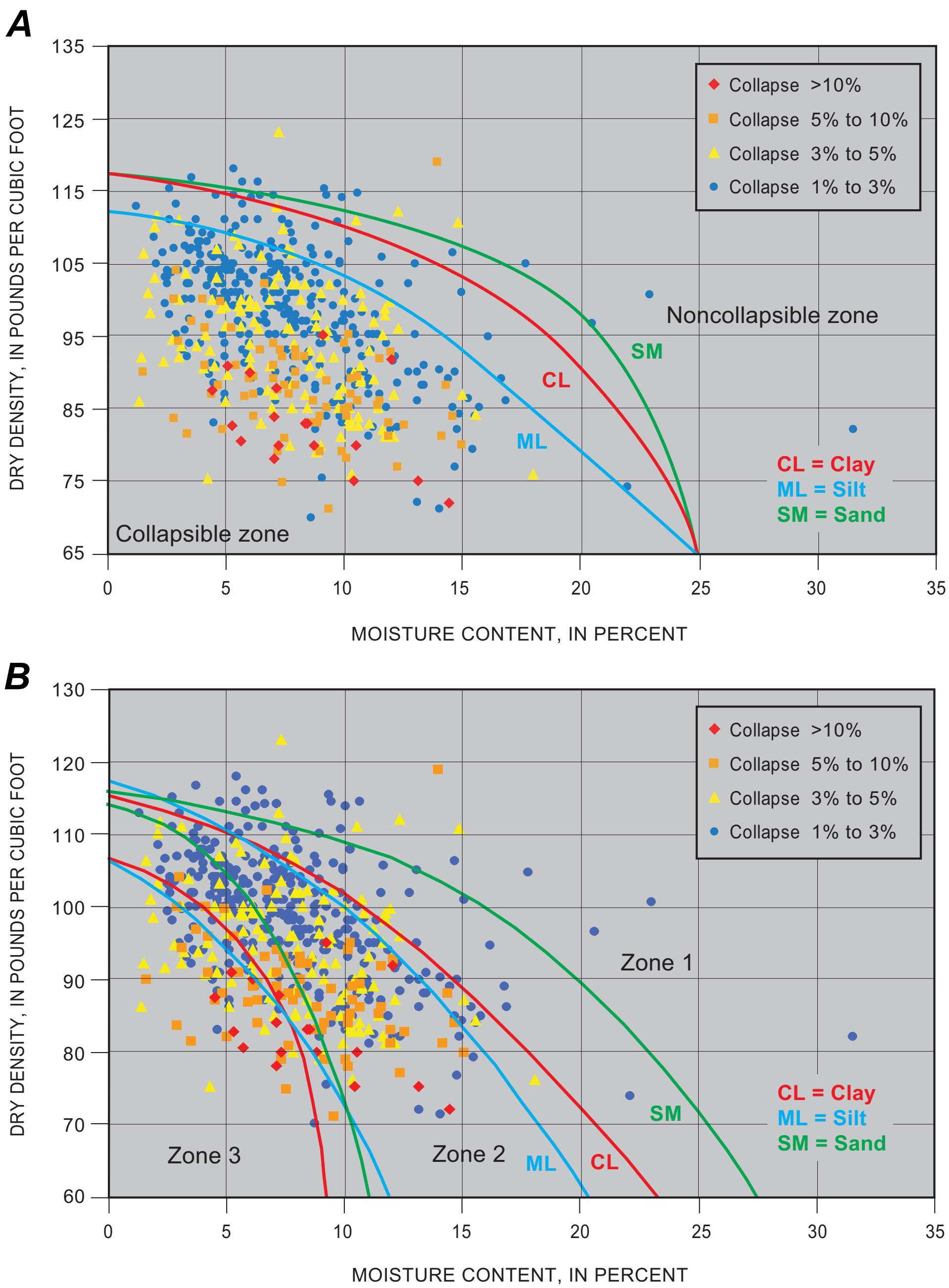


Figure 4-12. The relationship of dry density versus moisture content in statewide collapsible-soil swell/consolidation data with published collapse boundaries from

(*A*) Roullier and Stilley (1993) and

(*B*) Karakouzian and Roullier (1993). Data are from swell/consolidation test results when the soil is loaded to 1,000 psf and then wetted.

Collapse Zones	
Zone 1:	**0 to <1%**
Zone 2:	**1 to <5%**
Zone 3:	**<5%**

points. A number of conclusions can be made from this chart:

1. The boundary lines of the three collapse zones correlate well at high DD and very low MC.
2. Very few Colorado samples fall within the higher MC part of the chart where the soil-type boundaries of zone 1 and zone 2 are located.
3. For all three soil types, the high-collapse-probability lines for zone 3 do not suitably encompass all the high-collapse-potential test data from Colorado and exclude those samples having moderate DD values (about 90 pcf) and elevated moisture contents (i.e., MC as high as 10 to 15 percent).

Proposed Colorado Collapse-Susceptibility Chart

On the basis of the results of the Colorado statewide study, we propose another collapse-susceptibility version of the DD versus MC graph (**figure 4-13**). This new susceptibility chart for soils in Colorado is partitioned to indicate two zones—one of low-to-moderate collapse potential and one of moderate-to-high collapse potential—by using the collapse-severity categories of Mock and Pawlak (1983). These new boundary divisions are not adjusted for different soil classifications, but utilize a qualitative, best-fit curve to

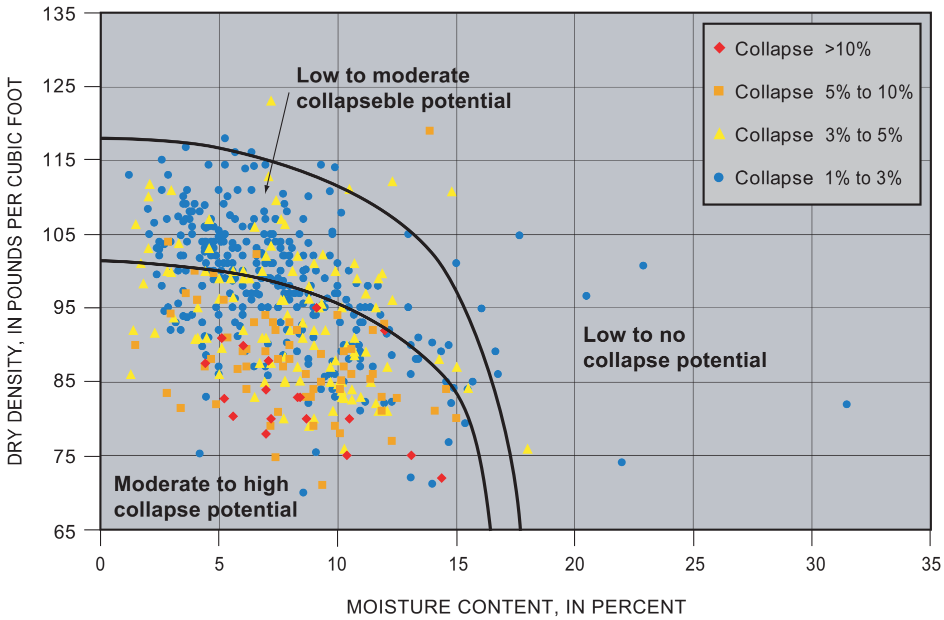


Figure 4-13. Proposed collapse-susceptibility boundaries based on dry density versus moisture content of soils in Colorado. Data are from swell/consolidation test results when the soil is loaded to 1,000 psf and then wetted.

encompass 96 percent of the test values that fall within the two susceptibility zones. The population scatter of Colorado test data was less than that of the Nevada data in Roullier and Stilley (1993) and Karakouzian and Roullier (1993), so the Colorado boundaries of collapse prediction show fewer data outliers and more realistically indicate levels of collapse probability. The limitations of consolidation testing that have been previously mentioned apply to this graph. Quality consolidometer test results are generally only available for finer-grained soils. It should be understood there is a scarcity of results from more granular sandy soils, and no results exist from gravelly soils, as sampling of these types of materials with thin-walled tubes is impractical or impossible.

The collapse predictions in figure 4-13 only reflect the behavior of dry, low-density soils that collapse upon wetting, as typically shown in the consolidometer-test chart in figure 4-3. Certain other types of lower-density but saturated clay and silt soils may plot to the right of the boundary lines and, although not exhibiting collapse as defined in this publication, may be highly compressive. Such examples of compressible soils include quick clays, organic soils, and lacustrine (lake) deposits not discussed in this publication. The important distinction is that collapsible soils, as defined and used here, settle quickly when water is added to the soil, which triggers the collapse of the soil skeletal structure. Soft and saturated compressible soils typically settle only under load when the additional weight causes the soil to expel or squeeze out the excess water, a process called soil consolidation.

Considerations with Coarse-Grained Collapsible Soils

Certain collapsible soils are very coarse grained; such soils can consist of greater than 75 percent gravel- and cobble-sized rocks. These soils occur in higher-energy depositional systems. Close examination of these gravelly soils reveals that the disseminated rock fragments are supported in a clayey-silt matrix, which could be collapsible. Geologic environment and geomorphology are key to determining the collapse susceptibility of coarse-grained soils (Rollins and others, 1994). Standard engineering index tests might not be very helpful, as these gravelly soils could have a dry density as high as 120 pcf. It also becomes very difficult to retrieve an "undisturbed" sample by using thin-walled tubes in these types of soils for typical swell/consolidation tests. Rollins and others (1994) stated that the CP may be lower, 1 percent to 4 percent, upon wetting at 1,000 psf, but collapse would be relatively rapid because of the permeability of the soil. A wetting front would propagate rapidly and deeply through the soil column in this case. Test pits or trenches would be best for observing near-surface soil structure (i.e., the spatial arrangement of the clasts in the finer-grained matrix), but the thickness of the collapsible gravel must also be known. For these types of soils, a more accurate assessment of future settlement would be obtained by in situ, field-wetting tests.

The Paradox of Expansive and Collapsible Soils

It is possible that a high-clay-content soil could exhibit both swell and collapse behavior. Many areas of western Colorado and along the Front Range are underlain by Cretaceous claystone, which is generally expansive. The derived alluvial silty-clay soil can have high expansive (smectite) clay percentages while still being quite dry and exhibiting the soil-fabric properties diagnostic of collapse. Unique to collapsible soils containing smectite clay is that they are expansive at very light loads upon wetting but have high shear strengths in a dry state (Bull, 1964). Lofgren (1969) noted that certain soils briefly expanded when wetted and then collapsed substantially. Barden and others (1973) described certain flood-plain alluvial clay soils in the Tucson, Arizona, area that had swelling-clay characteristics with high LL and PI properties; these soils expanded when wetted under low pressures (load), but then collapsed under higher pressures. Jennings and Knight (1975) mentioned this phenomenon and discussed a "region of heave" and a "region of collapse" at various overburden pressures when double consolidometer tests are conducted for the same soil. Wetting experiments of entire columns of collapsible soil by the New Mexico Bureau of Mines and Mineral Resources near Espanola, New Mexico, have revealed that as the wetting front moved laterally through the soil, the initial wetting of expansive clays could cause compressive forces, and the ground surface actually swelled up before it collapsed (Love and others, 1987, 1995). From the overburden pressures of the weight of the soil alone, collapse might not occur from simple wetting if the degree of saturation were less than what is required to disperse the agglomerations of clay platelets and allow the soil grains to shear and reorient in a denser configuration. However, applying load could induce collapse upon wetting and cause a very steep consolidation curve at further incremental loads in consolidation testing. As a result, clay-rich soils with these contrasting behaviors have been called hydrocompressive or hydroconsolidatible soils because they are subtly different from hydrocompactive soils.

There are case histories in western Colorado involving clay-rich alluvium substrate where heave occurred in a very lightly loaded concrete sidewalk and settlement occurred at an adjacent, more heavily loaded structure foundation when the area was wetted. Local geotechnical consultants have recognized this phenomenon and have adjusted their consolidation testing to wet the sample either at the onset of load or at light loads of 100 psf, 250 psf, or 500 psf. **Figure 4-14** is an example of a swell/consolidation test of a hydrocompressive soil that, when wetted, heaved at light loads. If this same sample were initially loaded to 1,000 psf and then wetted, substantial collapse would have occurred. Regional differences in swell/consolidation testing in Colorado are further discussed in Chapter 8.

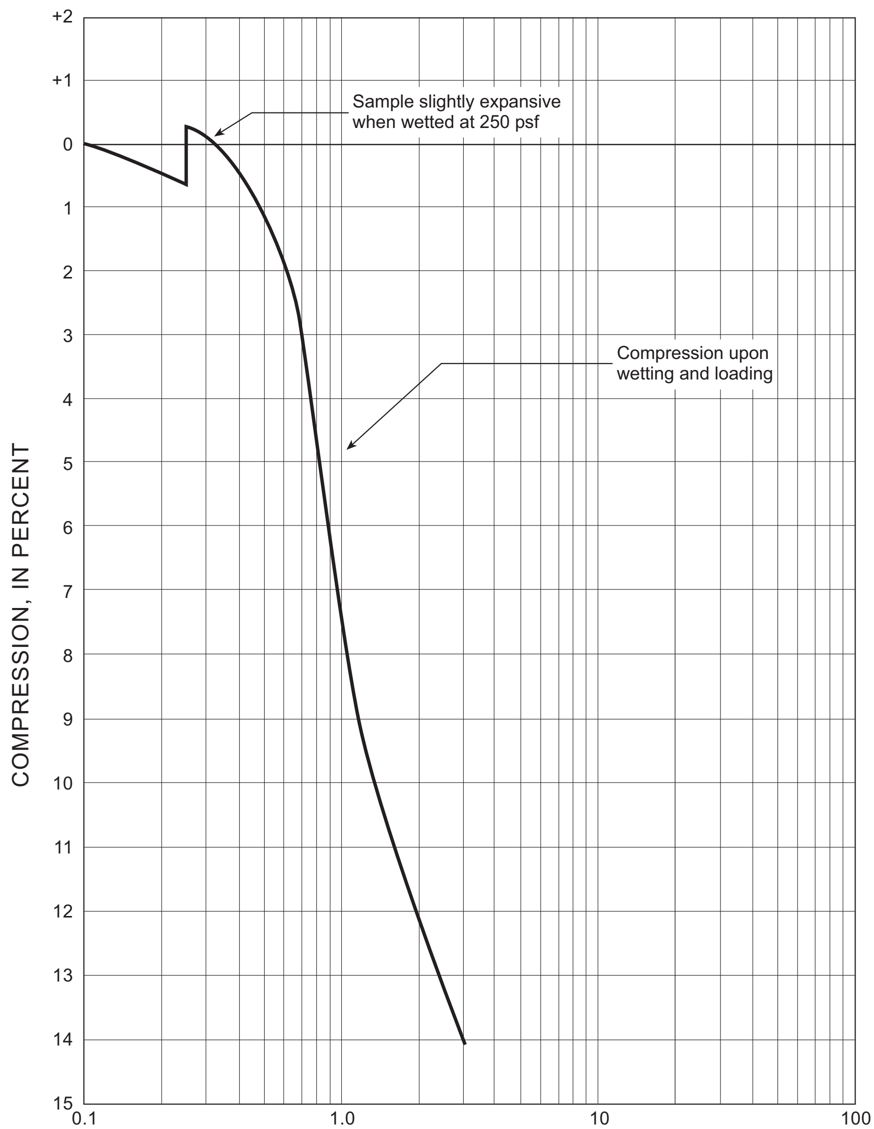


Figure 4-14. Swell/consolidation test example of a clayey soil that, when wetted, is expansive at very light loads but collapsible at higher loads.

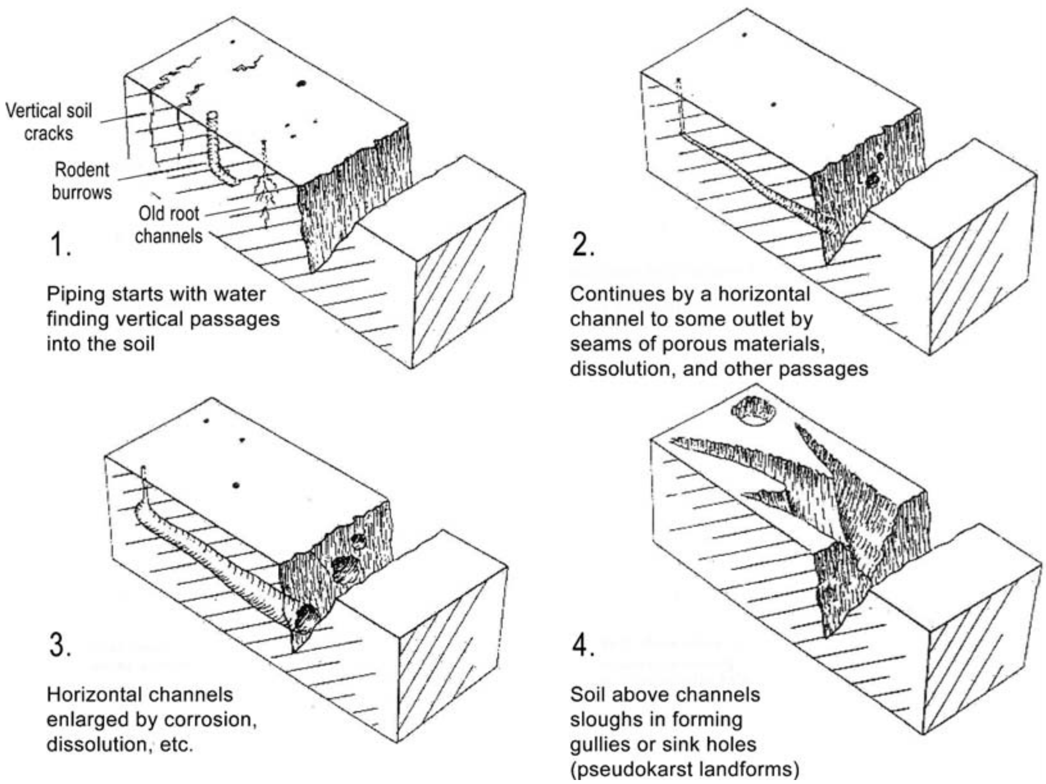


Figure 4-15. The evolution of soil pipes. Illustration modified from Cline and others (1967). A more detailed illustration of this erosional process and resultant land forms is shown in figure 3-5.

DISPERSIVE SOILS

Soil dispersion and piping are common in dryland areas. The process is considered such a serious problem in the Montrose area that the authors of the *Soil Survey of Delta-Montrose Area* commented at length about the phenomena and included the illustration shown here in **figure 4-15** (Cline and others, 1967). Dispersion susceptibility is a function of the clay mineralogy and the weakness of the electrochemical bonds of mostly smectite (montmorillonite) clay particles, which is governed by the higher ratios of sodium ions to calcium and magnesium ions (Heede, 1971; Parker and Jenne, 1967; Sherard and others, 1976a, 1976b; Parker and Higgins, 1990). Clay agglomerations with high sodium content more easily deflocculate in fresh water; in this process, the chemical bonds break and the clay particles disperse, to be washed away in suspension, even in low-velocity water. A number of dam failures, including one in Colorado, have been attributed to dispersion (Sherard and others, 1976b).

The standard tests for soil identification commonly seen in geotechnical reports (e.g., moisture content, density, sieve gradations, and Atterberg plasticity limits) are not capable of identifying dispersiveness in clay soils. Four tests for dispersive soil, investigated by Sherard and others (1976b) and found in NRCS (1991), are discussed here: (1) crumb test, (2) pinhole test, (3) double-hydrometer test, and (4) soluble salts in pore water and calculation of the sodium-absorption ratio (SAR). Most of these now have ASTM test guidelines.

Crumb Test

The crumb test is one of the easiest tests for making a qualitative determination of dispersion. A small sample, or crumb, of soil (¼ to ⅜ in. in size) is preserved at its natural moisture content. The sample is placed into a beaker of distilled water and observed for 5 to 10 min. The following interpretation guide is from Sherard and others (1976b):

- Grade 1. No reaction: Crumb may slake and develop as a flattened pile on the bottom of the beaker, but there is no sign of cloudy water (e.g., no clay colloids in suspension).
- Grade 2. Slight reaction: Just a hint of cloudy water near the surface of the crumb.
- Grade 3. Moderate reaction: Easily recog nizable cloud of colloids in suspension, usually spreading out in thin streaks from the crumb on the bottom of the beaker.
- Grade 4. Strong reaction: Colloidal cloud covers nearly the whole bottom of the beaker, usually in a very thin skin. In extremely dispersive crumbs, initial streamers of colloids can be seen, at times arcing from the crumb, and the entire water in the beaker can become cloudy.

Sherard and others (1976b) cautioned that the crumb test is a very good indicator of dispersive soils if the test is positive. The converse was not always true, however. In their work, 40 percent of the known dispersive soils that they sampled in their study had a nondispersive reaction in the crumb test. The crumb test is standardized by ASTM D 6572-00: *Standard Test Methods for Determining Dispersive Characteristics of Clayey Soils by the Crumb Test*.

Pinhole Test

In a pinhole test, a pinhole (1-mm diameter) is punched through a sleeved cylindrical clay soil sample that is placed in an apparatus through which a constant flow of distilled water is passed (Sherard and others, 1976a). If the water becomes colored and the hole rapidly enlarges, the soil sample is considered dispersive. If the water flow is clear and the hole does not enlarge, the sample is considered nondispersive. For more information on interpretation, the reader can review NRCS (1991) online. This test has been standardized by ASTM D4647-93(1998)e1: *Standard Test Method for Identification and Classification of Dispersive Clay Soils by the Pinhole Tes*

Double-Hydrometer Test

In this test, a particle-size distribution by hydrometer is performed on two equal-weight samples of the same soil that are placed in equal volumes of water. Sample A is considered the natural dispersed sample whereas sample B is subjected to strong mechanical agitation and a chemical dispersant. The dispersion of sample A will be smaller, so the difference in the percentage of fines between the two samples after the test procedure is called the percent dispersion. As the percentage of clay fines (0.005 mm or smaller) in sample A approaches that in sample B, the natural dispersion of A increases and the percent dispersion will increase (**See figure 4-16**). For example, if, after the test, the percentage of remaining fines in a hydrometer test of both A and B is equal, the percent dispersion is 100 percent, which is full dispersion with or without dispersants or strong agitation. The double-hydrometer test has been standardized by ASTM D4221-99: *Standard Test Method for Dispersive Characteristics of Clay Soil by Double Hydrometer*. The guidelines on interpretation of the double-hydrometer test are shown in **table 4-4** (NRCS, 1991).

Table 4-4. Interpretation of Double-Hydrometer Test.

Dispersion (%)	Interpretation
>60	The soil is probably dispersive.
<30	The soil is probably not dispersive.
Between 30 and 60	Other tests are needed.

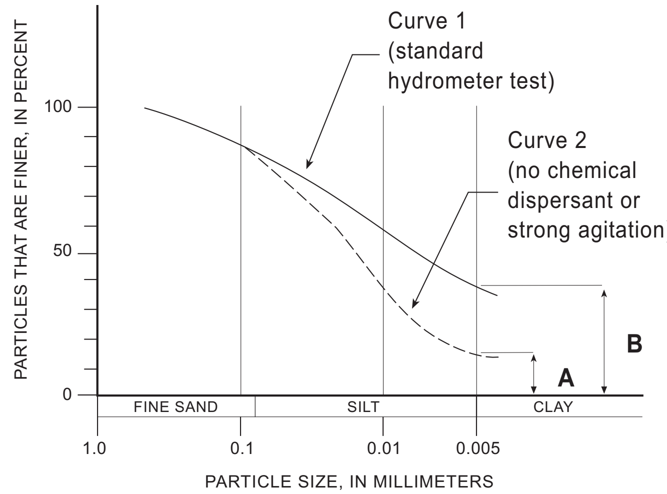


Figure 4-16. Sample double-hydrometer test result and calculation of percent dispersion (note log scale for x-axis). By definition, the percentage dispersion equals the ratio of A to B times 100%. Percentage A is the amount of clay released by the sample without dispersant or agitation. Percentage B is the percent clay indicated by the hydrometer test.

Soluble Salts in Pore Waters

There is a correlation between high sodium ion levels and increased dispersivity and piping of soils (Brown, 1962; Parker and Jenne, 1967; Heede, 1971). Soil scientists at the NRCS use standard laboratory tests that can be performed in a laboratory setting or in the field by using commercially available field soil-chemistry kits to measure soluble salts in the pore waters of soil samples. A soil saturation extract is prepared (i.e., a soil is crushed and saturated, the salts dissolve in the water, and this water is then extracted), and the water-soluble ions sodium, calcium, and magnesium are chemically measured in milliequivalents per liter (meq./liter). A sodium-absorption ratio (SAR) value is calculated by using the formula

$$SAR = \frac{Na}{\sqrt{\frac{1}{2}(Ca + Mg)}}$$

As the ratio of sodium to the other salt ions increases, the SAR value becomes larger. Much of the research by Sherard and others (1976b) compared experimental crumb tests, pinhole tests, and double-hydrometer tests (yielding percent dispersion) of samples of known dispersive clay soil to the respective measured sodium-absorption ratios and total dissolved salts in the saturation extract. By plotting pore-water chemistry data and the respective severity of dispersion of a particular soil, they developed an empirical, three-zoned chart based on the measured salts in sample pore waters (**figure 4-17**). The following are the descriptions for their three zones:

- Zone A. Almost all soils that fall in zone A are dispersive. This zone includes soils sampled from embankment dams that were damaged or failed because of dispersion and piping.

- Zone B. Soils with pore-water salts that fall into zone B are considered nondispersive and fall in the ordinary, erosion-resistant clay soil category. However, this zone can include very silty (ML) soils that can be erodible in a pinhole test. A low-density, hydrocompactive silt could fall into this category.

- Zone C. Soils in zone C can range from dispersive to nondispersive. Pinhole tests can indicate colloidal dispersion but at very low rates.

For most new NRCS soil surveys—such as the Ute Mountain area soil survey in Montezuma County, Colorado, and New Mexico—SAR is a standard index that is measured and listed in tables of soil properties (Doug Ramsey, personal commun., 2001).

GYPSIFEROUS SOILS

The collapse potential of gypsiferous soils in the Western United States has been known for some time by soil scientists at the NRCS. Nettleton and others (1982) described the subsidence of gypsiferous soils and damage to foundations, irrigation canals, and roadways. Actual properties of these types of soils were researched in-depth in Russia, where distress to structures occurred in certain arid to semiarid terrains of southern Russia and the former Soviet republics. Petrukhin (1989, 1994a, 1994b) and Mikheev and others (1977) described properties and soil types that are similar to those seen in southwestern Colorado and concluded that gypsiferous soils in their native condition are characterized by (1) negligible compression and very high strength in the original dry state, (2) decreasing strength during short-term wetting, and (3) different types of deformation depending on the type of soil (i.e., sand, sandy silt, sandy clay, or clay).

The literature record of collapsible soils in Russia made the distinction between settlement properties of (1) typical hydrocompactive soils with lower percentages of gypsum as a soil-cementing agent and (2) soils with higher percentages of gypsum exposed to longer-term leaching and suffosion (micropiping), which can show soil-porosity increases and longer-term settlements upon prolonged wetting. Referencing his earlier collaborative work in Mikheev and others (1977) measuring load-test settlements after experiments lasting as long as 160 days, Petrukhin (1989) wrote, "subsidence of sandy loams is occasioned by partial destruction of cementation links owing to the softening and dissolving of gypsum in the places of its contact with insoluble particles of soil." He made the distinction, though, between ground deformation from typical collapse, a short-term process, and the much longer term suffosion. Suffosion can result in the formation of macropores when the soils have undergone long periods of heavy wetting without any appreciable

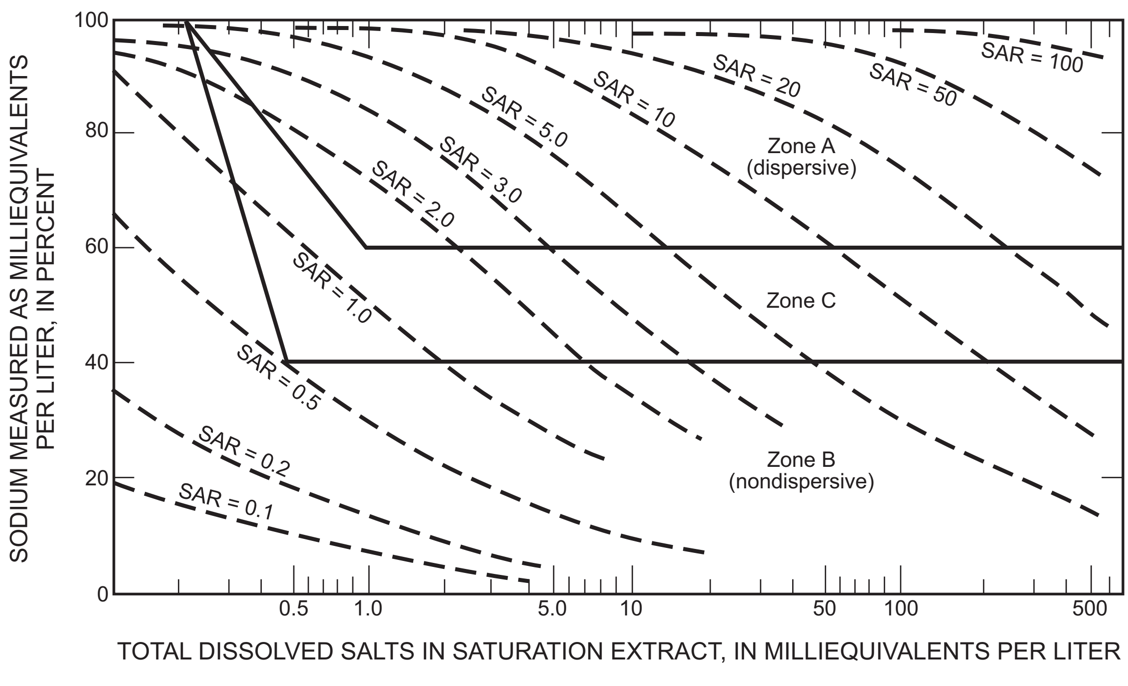


Figure 4-17. Comparison of pinhole-test results with soil chemistry sodium-absorption ratio (SAR) calculations. Illustration modified from Sherard and others (1976b). Used by permission.

loading. The presence of both gypsum cement and suffusion-caused soil-porosity increases has been identified in previously irrigated areas of clay-rich, gypsiferous soils in western Colorado. Consolidation testing of these types of soils can yield greater than 10 percent collapse at 1,000-psf loads (Laurie Hauptmann, personal commun., 2002).

Although a consolidometer test provides measurement of collapse due to softening and dissolving of gypsum cement, it does not provide any indication of the long-term risk of settlement caused by continued wetting. Only a measurement of percent gypsum in the soil can assist in evaluating that potential hazard. NRCS soil scientists use a fairly simple field chemistry method developed by the Hach Company to measure percent gypsum in a soil (Nelson and others, 1978). In the ethylenediaminetetraacetic acid (EDTA) titration method, each drop of EDTA solution needed to change the color of an extracted solution is approximately equal to 1 percent gypsum by weight. Thus the method provides an approximation of total percentage of gypsum by weight of a sample. Gypsum-rich soils, with their high sulfate content, are also corrosive to normal concrete and can be evaluated by sulfate tests commonly performed by geotechnical laboratories.

5. GEOMORPHOLOGY AND SOIL DEPOSITION

Much of Colorado has the necessary climate, source rocks, and topography for the generation of collapse-prone soils: (1) a semiarid environment characterized by windy conditions and intense thunderstorms, which quickly erode weak rock; (2) wide expanses of clay- and silt-rich, poorly indurated rock formations; and (3) adequate topographic relief above the depositional areas (e.g., the valley floors, basins, swales, drainageways, and other low-lying areas where sediments can accumulate) to allow rapid deposition in environments dominated by water processes. Wind deposits tend to blanket flatter areas, such as the leeward side of low hills, mesa tops, open plains, and Pleistocene pediment surfaces.

Arid to semiarid climates generally have much higher erosion rates and, subsequently, higher sediment yields (Langbein and Schumm, 1958) (**figure 5-1**). Semiarid areas have low vegetative cover, but are still exposed to episodes of intense thunderstorms capable of generating sufficient runoff to transport large amounts of sediment as debris flows. The peak sediment yields occur in areas where annual precipitation ranges from 8 to 18 in., which is typical for most areas of Colorado that are not in higher-elevation, mountainous zones. High sediment yields have created thick accumulations of Holocene deposits in many parts of Colorado.

As can be seen in figure 5-1, sediment yield drops off radically in very arid areas. Intense thunderstorms rarely occur in these areas, so sediment yield is minimal, even with a lack of vegetation. Conversely, sediment yields decrease gradually with increasing rainfall, as the climate becomes more moderate and erosion is reduced because of thicker vegetation covering the ground surface. The map of Colorado in **figure 5-2** shows the State's physiographic regions and the areas that exceed 18 in. of annual precipitation. Overlaying the case-history sites clearly shows a trend of collapsible-soil occurrences where annual precipitation is less than 18 in.; most locations fall into the 12- to 16-in. range. Although the location and concentration of case histories are somewhat skewed by topographic constraints and the abundance of information from populated or fast-growing areas of the State, this generalization is still basically sound—collapsible soils do not occur in high-elevation areas with annual precipitation greater than 18 in. In many areas of Colorado, gypsiferous or salt- or sulfate-rich shale bedrock is exposed at the surface, and plant cover is minimal (**figure 5-3**). Such sparsely covered terrain, much of which could be called badlands, is even more prone to erosion.

The surface or near surface of many areas of Colorado outside the central mountains is underlain by geologic formations that are of Late Cretaceous and Cenozoic age (pl. 1). During that geologic time frame, the paleoenvironment in Colorado transitioned from predominantly nearshore marine to terrestrial as the Rocky Mountains formed. The rock formations are mostly shale, mudstone, and sandstone. These younger formations are often poorly indurated, that is, they are not "hard." For example, crumbly claystone is

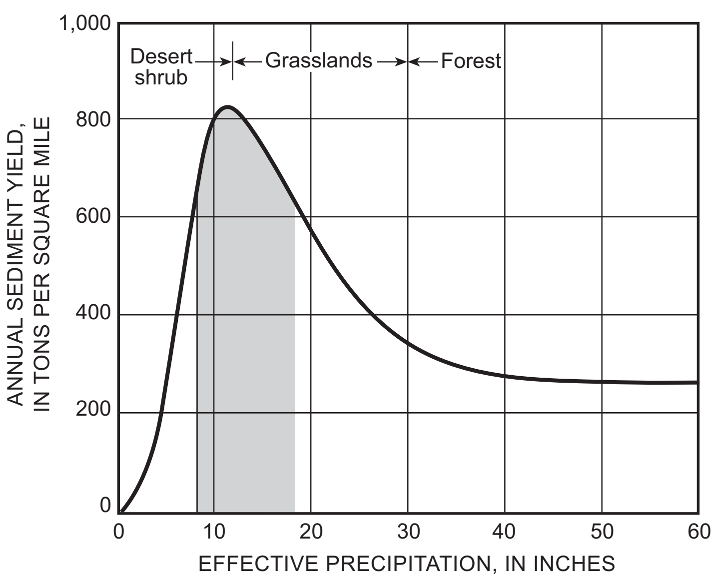


Figure 5-1. Annual sediment yield (erosion) based on annual precipitation, modified from Langbein and Schumm (1958). Shaded area is the range of annual precipitation typical for Colorado except for the high mountain regions.

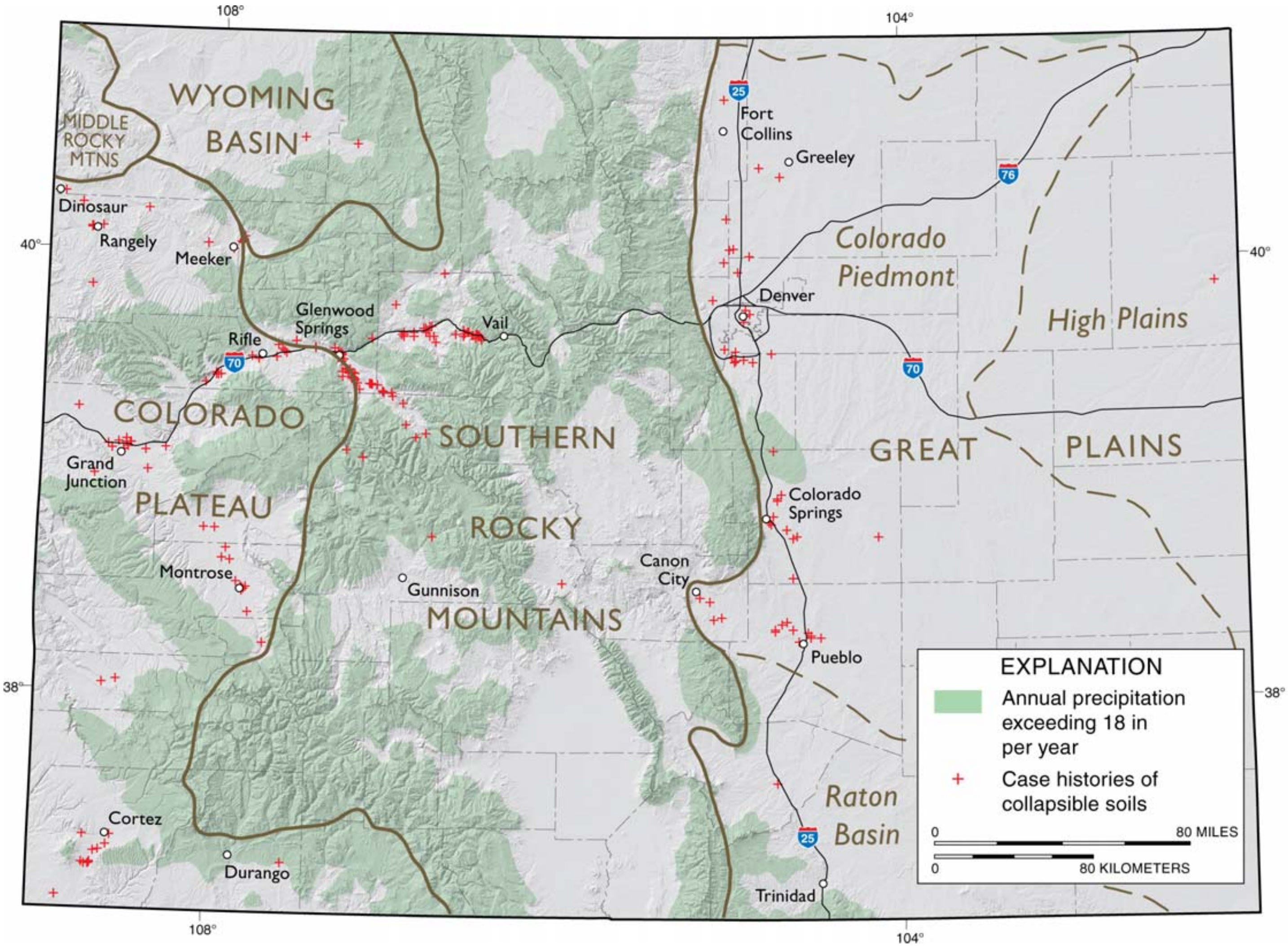


Figure 5-2. The State's physiographic regions and the areas of Colorado where the average annual precipitation exceeds 18 in. The locations of collapsible-soil case histories compiled for this study are plotted.

more common than hard fissile shale. Generally, the abbreviated burial diagenesis (the long-term compaction, cementation, and recrystallization processes as buried sediments become sedimentary rocks) of these formations has resulted in softer and easily erodible rocks. **Figure 5-4** shows an Interstate 70 highway cut in the highly erodible Tertiary Wasatch Formation of western Colorado. The two photographs illustrate the degree of rilling, gullying, and piping erosion that has occurred in the bedrock since the cut slope was excavated in the mid-1970s.

The research for this Colorado study included examination of the local geology, geomorphic terrains, and soil deposits at Colorado-specific case-history locations where collapsible soils were known to occur. This compilation verified that certain types of geologically recent (Holocene) surficial deposits are prone to soil collapse. This publication primarily examines Holocene and late Pleistocene deposits, generally no older than Late Wisconsin (or Pinedale) Glacial ages (generally younger than 15,000 yr). The following geomorphic settings, deposits, and rock types are associated with various forms of collapsible soils:

- alluvial fans and debris-flow fans,
- colluvial slopes,
- fluvial flood-plain and overbank deposits,
- eolian deposits (loess),
- gypsiferous soils and soils derived from evaporite bedrock, and
- near-surface weathering and alteration of gypsiferous Mancos Shale.

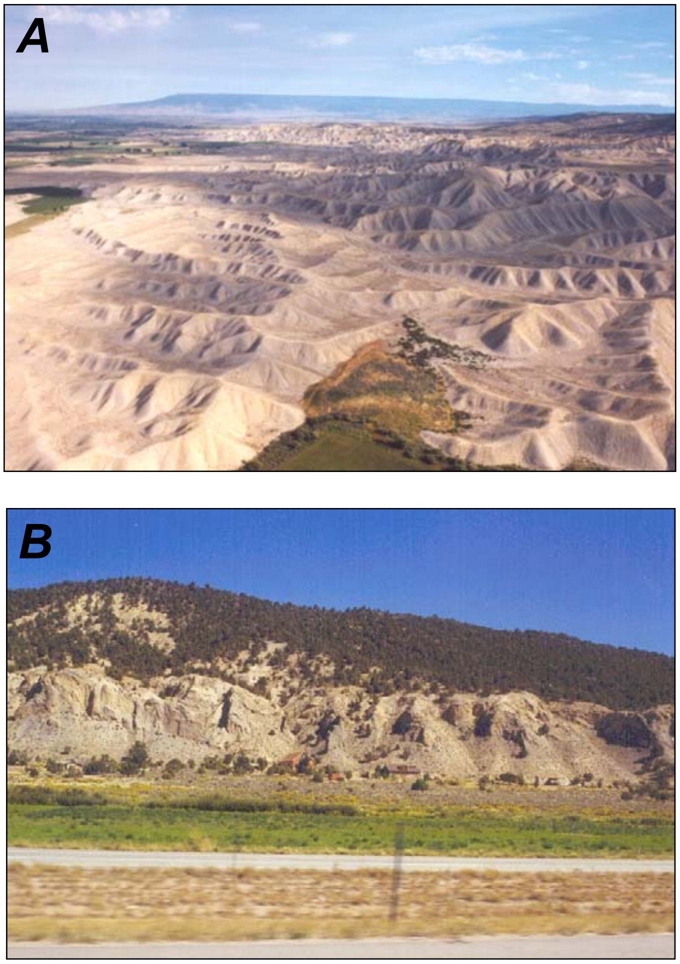


Figure 5-3A, Bare hills of Mancos Shale between Montrose and Delta. Grand Mesa is in the background. B, Poorly vegetated hills of evaporite bedrock in the Roaring Fork River Valley between Carbondale and Basalt; almost all the buildings at the base of the slope have been damaged by ground settlements.

Figure 5-5A is a conceptual block diagram illustrating the various landforms and surficial deposits that are mentioned in this publication. An actual digital elevation model from the Roaring Fork Valley near Glenwood Springs (**figure 5-5B**; White, 2002) also illustrates the topography and geomorphology that can result in the formation of collapsible soils.

ALLUVIAL FANS AND DEBRIS FANS

Alluvial fans and debris-flow fans are very common in western Colorado, as they are in most Western States. Most mature river systems have their lower valley walls mantled by individual and coalesced fans at the mouths of ephemeral tributary streams.

Alluvial-fan morphology has been recognized as one of the primary landforms associated with collapsible soils in the Western United States (Lofgren,

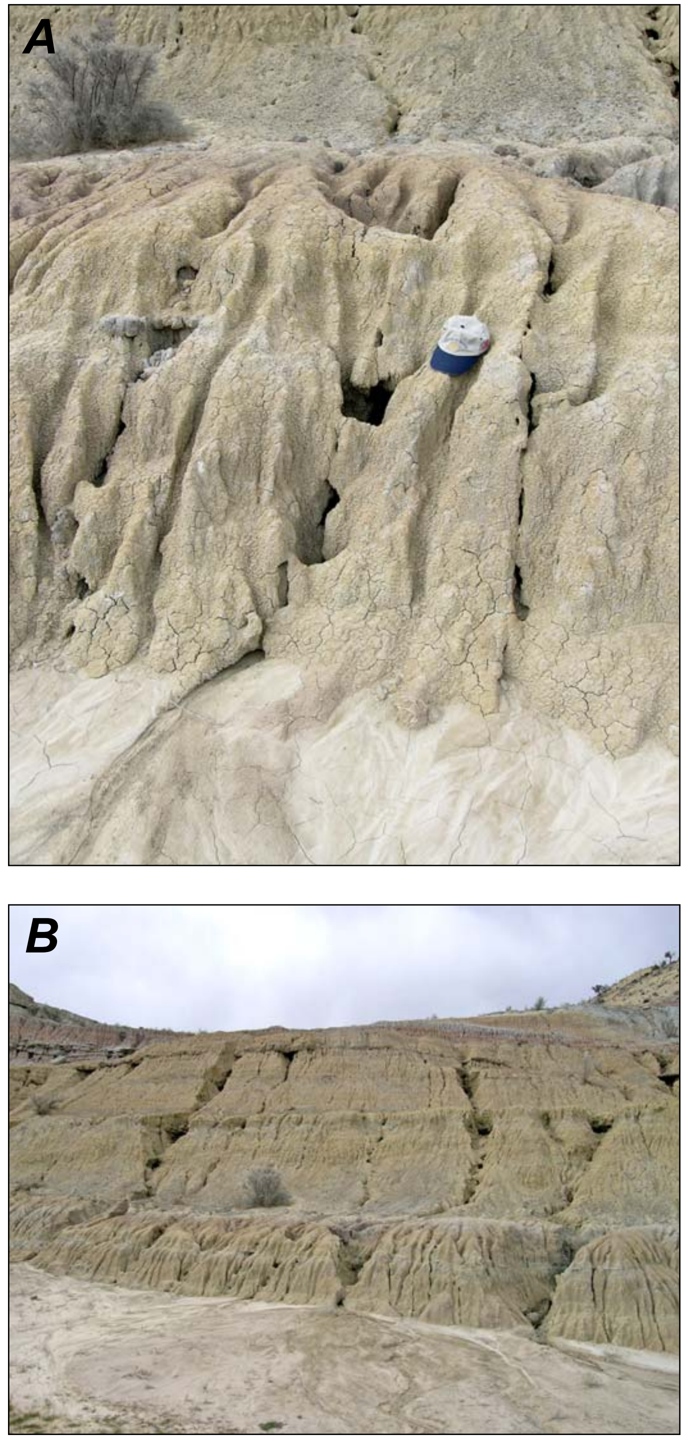


Figure 5-4. Extensive erosion in weak rock formations. This road cut, excavated in the late 1970s, is in the Tertiary Wasatch Formation, a series of poorly indurated claystone, siltstone, and sandstone widely exposed in western Colorado. Note extensive rills and piping voids and formation of micro–alluvial fans on the ditch floor. A baseball cap is shown for scale in *A*; the width of the ditch floor in the broader view shown in *B* is approximately 20 ft.

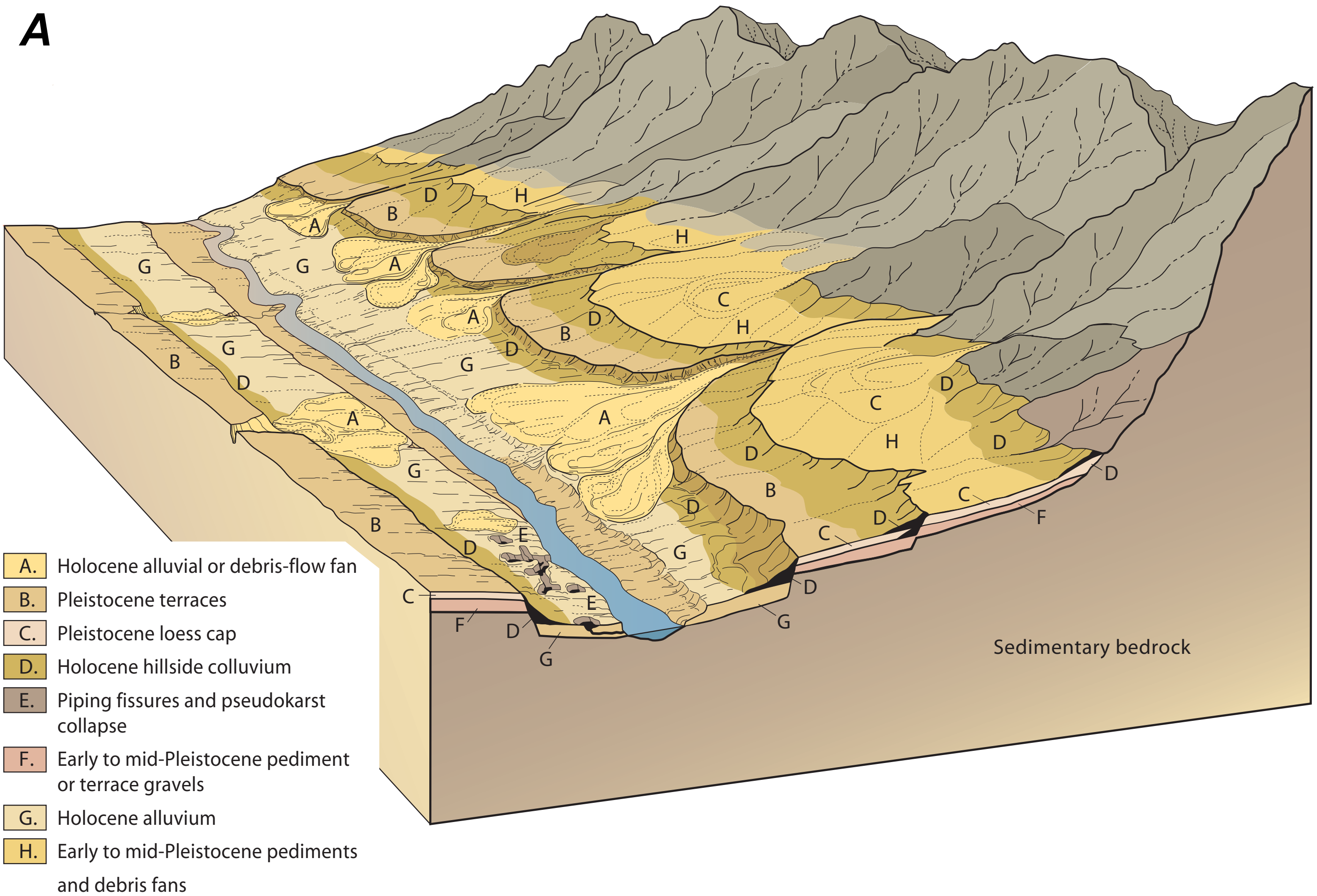


Figure 5-5. *A*, Geomorphologic landforms typical of western Colorado. Modified from Beckwith and Hansen (1989).

1960; Bull, 1964; Beckwith and Hansen, 1989; Rollins and Williams, 1991; Rollins and others, 1992a; Reimers, 1986; Luehring, 1988; Mock and Pawlak, 1983). Lofgren (1960, 1969) studied shallow subsidence due to wetting of alluvial-fan soils in California's San Joaquin valley. The following conclusions from that work are also applicable to Colorado:

1. Collapsible soils are found at arid margins of valleys where average annual rainfall is insufficient to penetrate below the root zone

2. The collapsible terrain seems to be only in Holocene alluvial-fan deposits.

3. Deposits are characteristically derived from short, steep ephemeral drainage systems that produce rapid runoff and low moisture pene tration.

4. Mudflow deposits are the overwhelming sedi ment type, but more sandy and gravelly deposits and interdepositional eolian deposits may also occur.

5. Alluvial-fan deposits that collapse upon wetting are typically of low moisture content and low density and have never been exposed to appreciable postdepositional wetting or saturation.
6. Collapse of these soils from wetting typically results in a volume decrease and density increase.

In a compilation and analysis of work in Utah, Rollins and others (1992a) summarized the geologic conditions conducive to formation of collapsible soils in alluvial fans: (1) arid to semiarid climatic conditions; (2) drainage basin composed of shales, mudstones, and siltstones; (3) relatively small drainage basin with poor

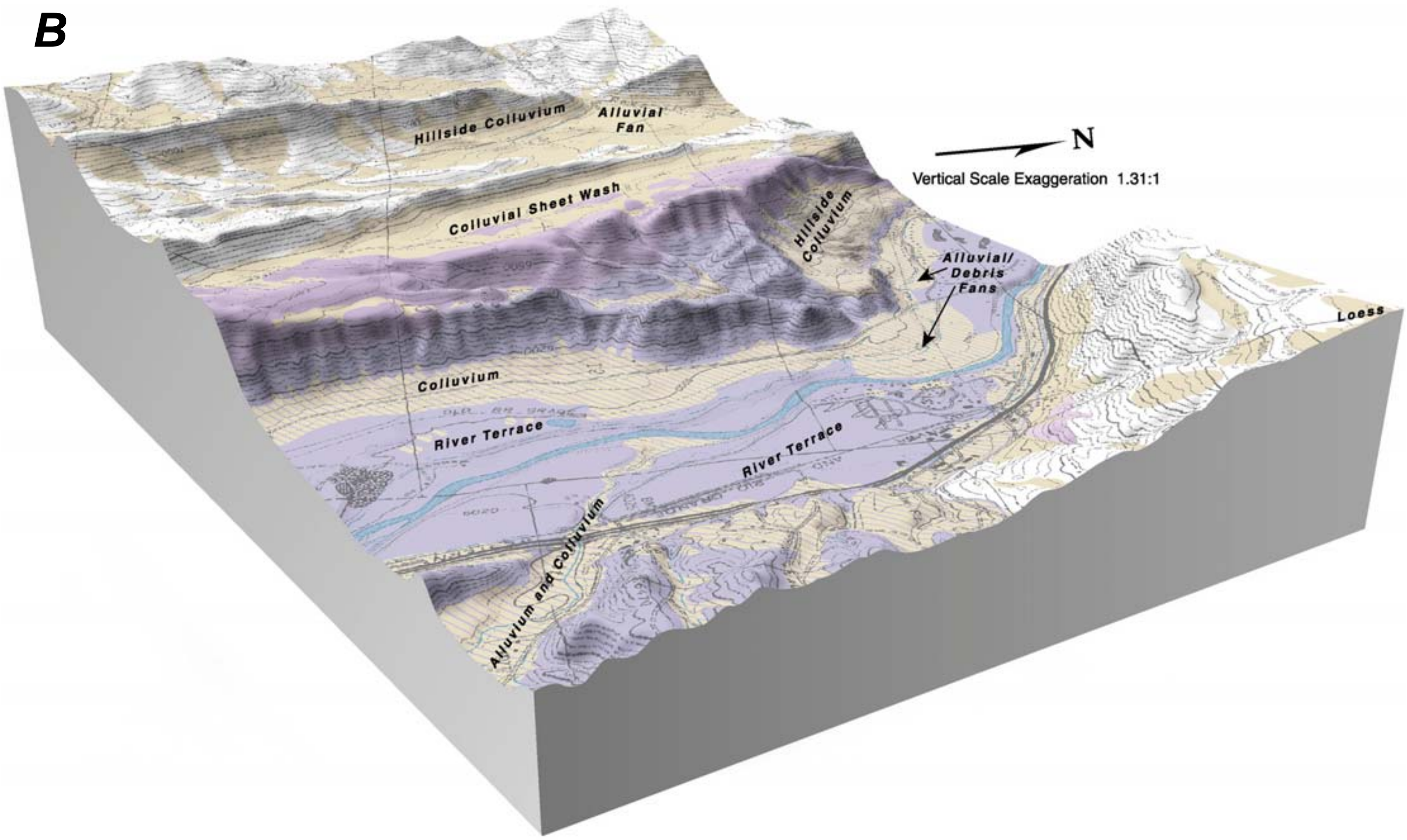


Figure 5-5 *B*, Block diagram generated from a 10-m digital terrain model with collapse-susceptible soil units mapped with tan shading and labeled. Blue and violet zones are areas underlain by evaporite bedrock, which will be discussed later in this chapter. Location is in the Roaring Fork River valley approximately 6 miles upriver from Glenwood Springs. Image from White (2002).

vegetal cover; (4) high ratio of alluvial-fan area to basin-source area; (5) mud- and debris-flow depositional environment on a Holocene to late Pleistocene alluvial fan; (6) significant depth to ground-water table so soils remain unsaturated and of low moisture content; and (7) derived soils composed of silts, sands, and low-plasticity clays. These conditions also pertain to Colorado.

Alluvial fans have a highly variable and complex stratigraphy. Each stratum, ranging from only inches to several feet in thickness, records one flooding event onto the fan. Because a flooding event and resultant sediment deposition might cover only a small part of the fan, the stratigraphy can be highly irregular, and individual strata or deposits might be lobate or sinuous in shape, having only limited areal extent. Sediments within each stratum can range in size from silt and clay to sand or very coarse debris. Individual flow characteristics can vary widely because they are dependent on rainfall intensity and duration as well as the combination of basin and slope geometry factors that, in turn, affect the hydraulics and the size fraction of sediments eroded from the slope. The alluvial fan is created by the slow accumulation of random sediment deposition that spreads from the mouth of an ephemeral stream onto a flatter slope area. Thicker and more granular deposits lie at the apex of the fan, in the steeper-gradient areas near the mouth of the upslope drainage channel. Finer sediments occur at the distal parts of the fan where accumulations are thinner. Fans in western Colorado can range from many tens of feet thick to well over a hundred. Examples of alluvial fans that might contain collapsible soils along the Colorado River corridor are shown in **figure 5-6**. In investigations of hydrocompactive soils that were encountered in broad alluvial fans and interfan deposits along the planned Interstate 70 alignment west of Rifle, Colorado, Shelton and others (1977) made some important observations:

1. The soils most susceptible to hydrocompaction were the interfan deposits of finer-grained materials

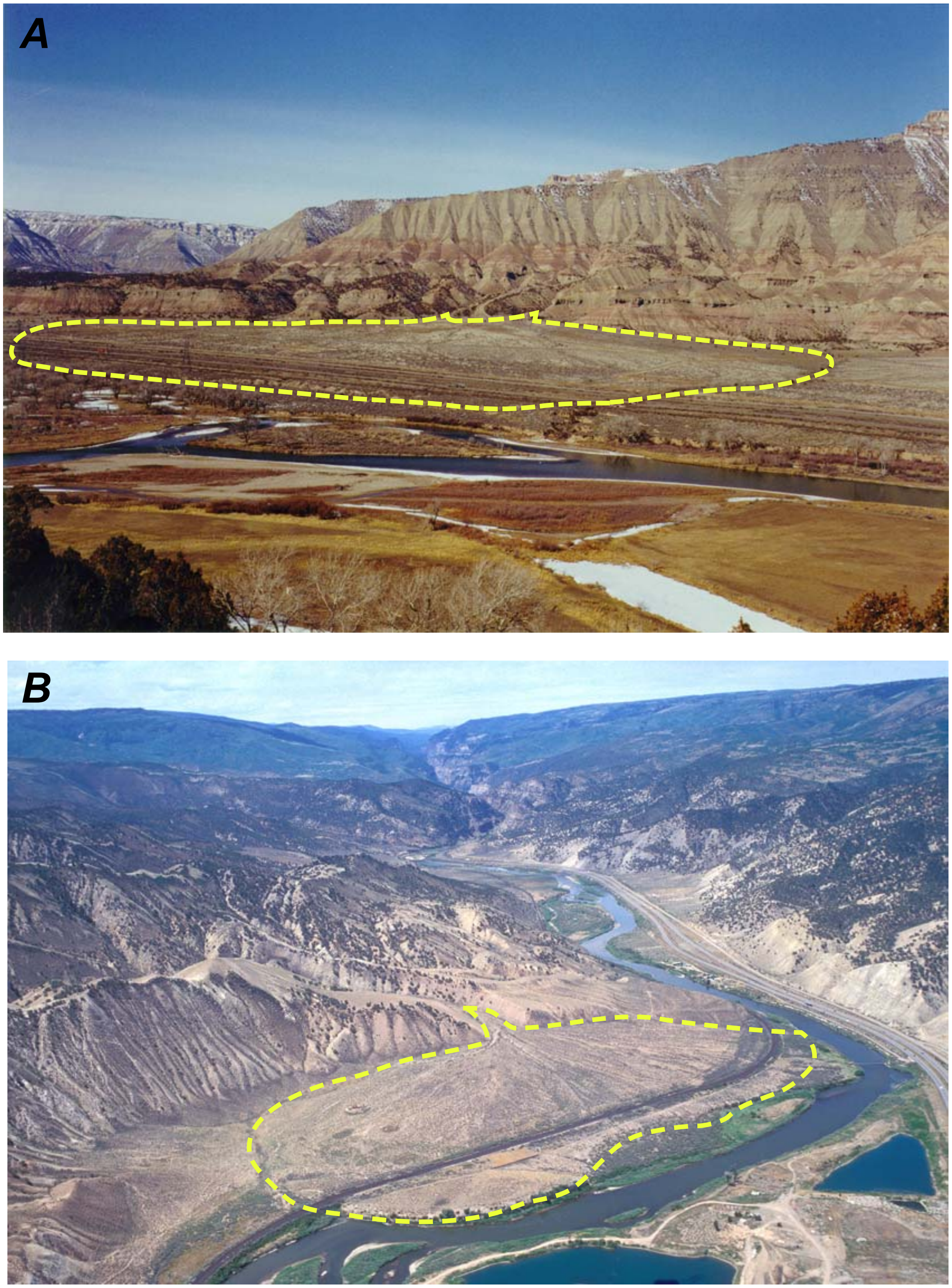


Figure 5-6*A*, A large alluvial fan in the wide Colorado River valley between Rifle and Parachute. The bluffs are the Roan Cliffs, composed of Wasatch and Green River Formations. *B*, A smaller alluvial fan near Dotsero, just below the confluence of the Eagle and Colorado Rivers in the Eagle Valley Formation. The rail-road alignment skirts the distal edges of several fans before entering Glenwood Canyon shown in the background. Individual fans are shown by dashed line.

2. Mudflow deposits in semiarid climates are likely to collapse when wetted.

3. Older deposits (predating Holocene and late Pleistocene deposits) are less susceptible to collapse than younger deposits.

4. Those deposits nearer to the change in gradient that causes the deposition (i.e., nearer to the source area) are more likely to collapse than those materials farther down the drainage.

5. The most reliable test is the plate load test during in situ flooding and saturation.

For the most part, Colorado does not have active tectonism along mountain faults where alluvial-fan sediments accumulate (such as the Wasatch Range east of Salt Lake City and the interior mountain ranges in south-central California). Where such active tectonic regimes do exist, for example, in the Rio Grande Rift System, the Arkansas Rift Valley, and the eastern boundary of the San Luis Basin (Widmann and others, 2002), the sediments in the alluvial fans along the range fronts are either in higher elevations and wetter climates or are generally not derived from clay- and silt-rich sedimentary rocks. The resulting coarser-grained sediments do not appear to be susceptible to collapse.

In arid environments, debris-flow and mud-flow deposits quickly dewater and then desiccate. The subsequent flooding and desiccation of the area do not allow deeper-seated wetting and the mechanical resorting and consolidation of the material that would happen in a fluvial (river) system. This condition leaves a series of deposits that have porosity composed of both microscopic and macroscopic-type voids. Bull (1964) recognized that voids are created in alluvial-fan sediments by many methods:

1. Quick cessation of vigorous flow causes the chaotic and randomly placed soil particles to remain in an open and precarious skeletal framework after the deposit has desiccated

2. Bubble cavities from entrained air in vigorous debris flows remain after drying of the mate rial (see SEM image in fig. 3-2).

3. Interlaminated voids are trapped in the roughly layered sediments.

4. Buried and unfilled mud cracks and soil fissures remain (**figure 5-7**).

5. Voids left in the soil from the decay and disin tegration of buried organic debris within the flow (**figure 5-8**) are subsequently covered by later debris or mudflow sediments.

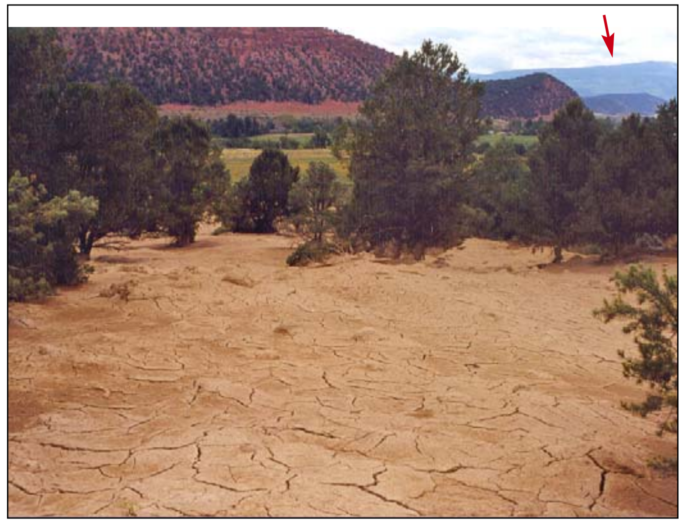


Figure 5-7. Mud cracks formed shortly after a 1997 debris flow in the Roaring Fork River valley. Red beds in middle background are the Maroon Formation. Basalt Mountain is indicated by an arrow.

Figure 5-8. Two alluvial-fan lobes of more viscous debris in another location of the same flow shown in figure 5-7. Note amount of vegetation debris in the lobes and the extent of ground cover that was buried.

Another major factor in the formation of collapsible soils in alluvial-fan deposits is the inherent stratigraphy of the deposit, both with regard to sediment type and thickness. Interbedded lenses and thin cobbly and gravelly layers commonly occur in finer-grained alluvial-fan deposits. These coarse deposits are formed from higher-energy single-event debris-flow floods and are never laterally continuous or homogeneous along the alluvial-fan surface. The coarser fraction of the flow settles out either as lobate debris-flow toes or sinuous lateral levees and channel fill. Upon further burial by succeeding finer mud-flow sediments, the gravel deposits stratigraphically pinch out down gradient into the surrounding finer-grained, less permeable, classically hydrocompactive sediments within the alluvial fan. These more permeable gravelly deposits can then act as down-gradient conduits for ground water during abnormal wetting episodes, actually creating a hydraulic head and a plume of subsurface water. This hydraulic condition allows further longer-term saturation, and deeper, more extensive migration of a wetting front into the dry collapsible soils in the vicinity. On high and dry alluvial fans, deep-seated wetting from broken water mains or sewer systems can induce settlement several hundred feet down the slope if water moves within buried, more permeable, gravelly layers.

Alluvial-fan morphology can create thick deposits of potentially collapsible soils toward the apex of the fan, near the mouth of an ephemeral stream. With a thicker collapsible-soil column, even small percentages of soil collapse (i.e., a collapse potential of less than 3 percent is considered low hazard) can result in adverse and potentially damaging settlement at the surface if a suitable column of soil were to saturate.

Western Colorado has abundant areas that are underlain by clay- and silt-rich rock formations, predominantly poorly indurated, friable Mesozoic and Cenozoic formations (see pl. 1). Bull (1964) stated that generation of alluvial-fan sediments with hydrocompactive properties occurs from basin source areas with clay-rich lithologies and indicated that soils with 12 percent clay content had the highest compaction upon wetting under a simulated overburden load. Very coarse, rocky debris materials in alluvial fans can collapse or settle when wetted and loaded if the soil is matrix supported. Work by Rollins and others (1994) revealed that collapsible rocky soils can even be derived from igneous rock. Rollins and others (1994) stated that the general requirements for potential collapse are dry soils with 5 to 30 percent fine-grained (silt and clay) material and matrix support of the larger soil clasts. No case history compiled for this publication indicates a collapse occurrence in soil derived from igneous rocks in Colorado, but that negative evidence may be due to igneous rocks' being more prevalent at higher elevations where the annual precipitation is higher.

Alluvial fans in western Colorado in arid to semiarid areas should always be suspect for collapsible soils. Several towns of western Colorado lie completely, or partially, on alluvial fans, including Glenwood Springs, Meeker, Rangely, Gypsum, Edwards, and Basalt. These towns all have experienced damage to structures due to collapse and settlement in alluvial-fan soils. More discussion on those specific areas is in Chapter 7.

COLLUVIAL SLOPES

Colluvial sheetwash and the slopes and soils that are created by these processes are closely related to alluvial fans. These slopes are formed at the base of hills or valley walls and drainage swales in response to erosion. If the highlands are steep and rock exposures are prevalent, talus slopes or gravelly colluvial soils can occur. Where the bedrock is fine grained and poorly cemented, slopes of fine-grained soils will develop. Colluvial sediments can be deposited as sheetwash, which is a blanket deposit eroded from a slope but not confined to a drainage channel. Beds may be roughly stratified but are generally poorly sorted, with gravel- to cobble-sized rock fragments that are matrix supported. Even very coarse colluvium, if matrix supported, can be collapsible (**figure 5-9**). These soil aprons or wedges can connect individual alluvial fans along the base of valley walls or mantle the interior swales of ephemeral drainage basins. Colluvial soils have many of the characteristics of finer-grained alluvial-fan soils and also have similar engineering properties.

Figure 5-9. Excavation in a colluvial slope in Glenwood Springs. This soil deposit is still considered susceptible to collapse because the gravel- and cobble-sized rocks are dispersed in a fine-grained collapsible matrix.

RECENT (HOLOCENE) FLOOD PLAINS

Holocene flood plains are created by the recent (less than 10,000 years before present) aggradation of alluvial sediments in river valleys and tributaries (**figure 5-10**). In earlier geologic mapping along the flank of the Front Range in the Colorado Piedmont (discussed in Chapter 7), these resulting Holocene sediments have been named the pre-Piney Creek, Piney Creek, and post–Piney Creek Alluviums. The interpretation of very young ages is based on the deposits' elevation above the current stream and their immature soil development. Where the older late Pleistocene (Pinedale) (older than 10,000 yr) glacio-fluvial gravelly and cobbly terrace surfaces are present, the Holocene deposits are located at a lower elevation, near the existing creek level. More often, in the dissected plateau or plains regions of Colorado, the Holocene deposits exist as the only terrace along second- and third-order tributaries and shallow washes. In these areas, the sediments are often derived from local clay-rich bedrock and sediment sources. Some of the fine-grained alluvium may be composed of reworked

Figure 5-10*A*. Typical tributary-creek alluvium in arid to semiarid Colorado; Oblique aerial view of North Mamm Creek flood plain.

Figure 5-10*B*. Typical tributary-creek alluvium in arid to semiarid Colorado; Oblique aerial view of arroyo incising Holocene flood-plain terrace east of Delta.

Figure 5-10*C*. Typical tributary-creek alluvium in arid to semiarid Colorado; Stinking Water Creek arroyo near Rangely.

Figure 5-10*D*. Typical tributary-creek alluvium in arid to semiarid Colorado; Arroyo behind business park near Pueblo Memorial Airport.

windblown loess, especially where Pleistocene loess remnants mantle some of the nearby mesas and pediments. In many locales these fluvial surfaces slope gradually up to colluvial pediments at the base of the hillsides that flank the drainage. In addition to aggraded alluvial-valley fill, recent flood plains also include broad mud flats, called "playas," and topographically subdued, but very broad, incised, alluvial fans of silty clay that blanket wide areas of low-lying ground between eroded and subdued hills of claystone and shale.

Recent flood-plain deposits characteristically have high clay and silt content and high salt and sulfate content (Soule and Stover, 1985; Cline and others, 1967). The vegetation is generally sparse. Recent alluvium and overbank deposits are commonly prone to dispersion, piping erosion, and formation of arroyos and pseudokarst landforms. Heede (1971) made important geomorphic observations about these arroyos: Nonpiping side slopes have a more gentle gradient and are well covered by vegetation. In contrast, piping gully side slopes are steep, unstable, have a "sugary" soil surface, and usually have no or sometimes very sparse vegetation.

Pseudokarst-type soil collapse is a very serious problem for development near arroyos where additional moisture can be introduced and water flow might not be controlled. In these types of soils, spontaneous formation of sinkholes and fissures is a common problem in roadways and irrigated fields, as well as in residential yards and lawns that are sited too close to an arroyo. Water runoff from impermeable pavements and culvert crossings can generate concentrated flows that accelerate soil dispersion and piping erosion. This is a real concern for maintenance of highways and control of headward gullying and erosion into road grades. Several sites from the Delta-Montrose, Silt-Rifle, and Cortez areas have piping problems that have affected, or are affecting, engineered works (**figure 5-11**). **Figure 5-12**, a photograph of severe piping erosion from poor control of field irrigation, shows an excellent example of the type of terrain that can quickly develop in these types of soils. The broken ground, open fissures, and soil sinkholes at the surface typify pseudokarst morphology, illustrated in figure 3-5 from Parker and Jenne (1967). Immediate soil collapse into large voids formed by piping, without warning, can be dangerous. Animals have been found dead in small collapse features similar to that shown in figure 3-6. Reportedly, heavy agricultural and construction equipment has suddenly, without warning or indication, dropped into such voids after their operators inadvertently drove onto a thin soil bridge capping an unknown subsurface void (**figure 5-13**):

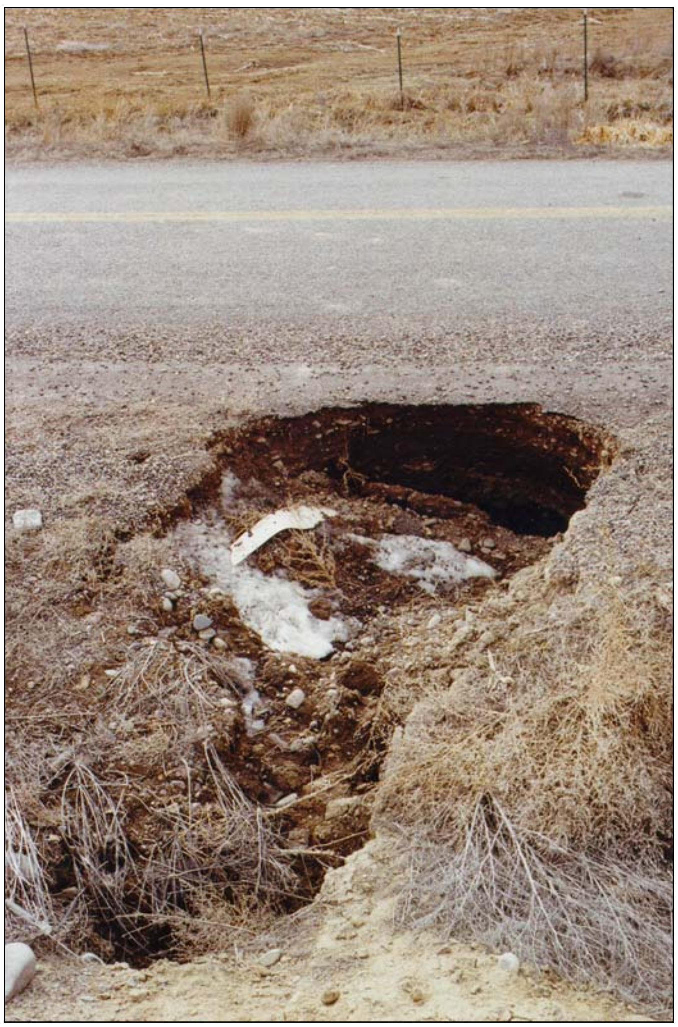


Figure 5-11. Dispersion and piping of soil near Loutsenhizer Arroyo led to this collapse hole along the shoulder of a Montrose County road.

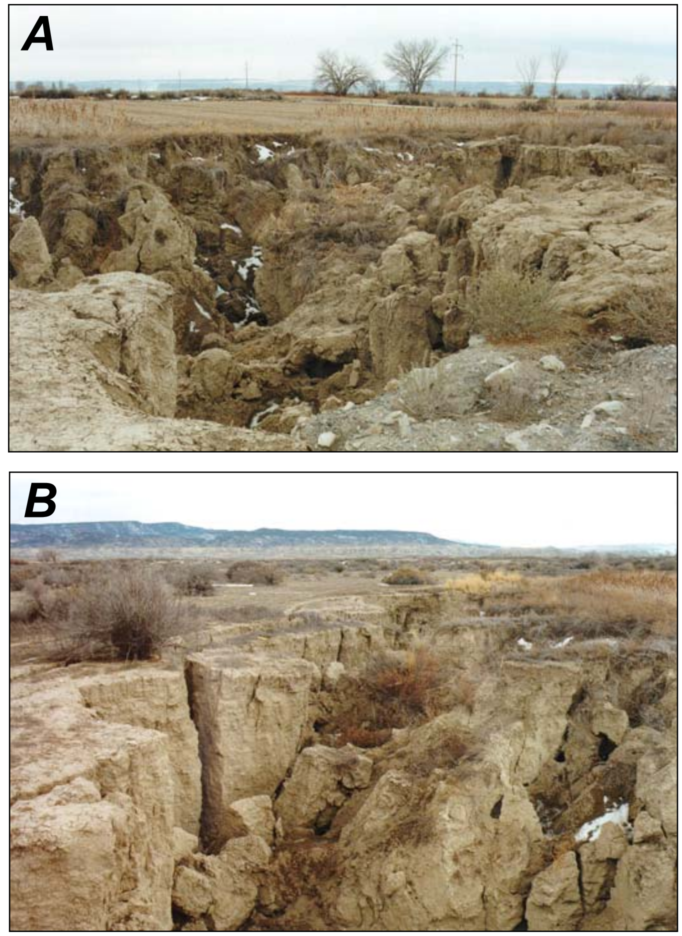


Figure 5-12. Erosion damage caused by piping that may occur from uncontrolled flows of water. This pseudokarst terrain was the result of irrigation runoff from an adjacent field near Loutsenhizer Arroyo, north of Montrose.

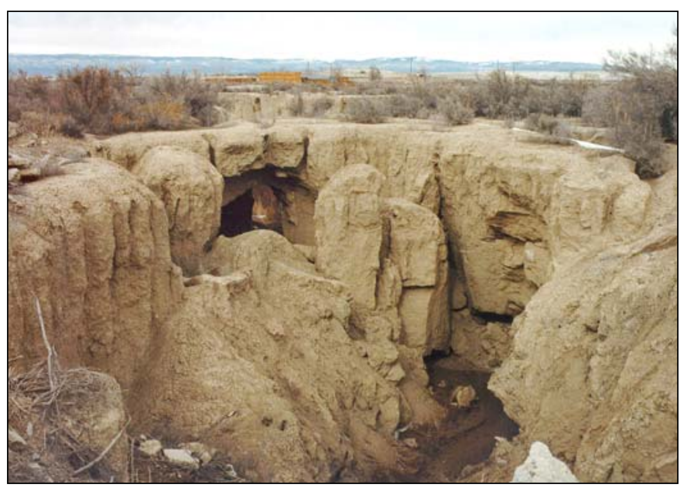


Figure 5-13. Soil dispersion and piping voids may create soil bridges. North of Montrose, progressive collapses of a large piping void, which had become essentially a subterranean drainageway, has created a tributary of Loutsenhizer Arroyo that is spanned by the soil bridge, which is about 8 ft long.

In arid areas, the soils found in recent flood plains also have the ability to hydrocompress if the clay content appreciably exceeds the silt content. Upon wetting and additional loading, consolidation of the saturated soils may be long term, such that settlement is not completed immediately but is continual. Soil strength may be reduced to the point that there is an actual loss of bearing capacity as the soil fails and begins to shear.

EOLIAN DEPOSITS

Eolian deposits are sediments that have been deposited by wind. Dune sand is composed predominantly of sand-sized grains. Windblown silt and clay deposits are called loess, from a German term for "loose" soil, which was first recognized along the Rhine valley. Eolian sediments are derived primarily from wind scour and deflation of river flood plains and secondarily from direct ablation of exposed rock formations, especially those that are softer. For a flood plain to be a significant source of eolian deposits, it must be wide relative to the depth of the valley and (or) carry a large sediment load. The particle size of eolian deposits is dependent on the source material and the processes that were prevalent in the recent geologic history. Eolian sand dunes (hills) and related wind-scour deflation structures are oriented according to the prevalent wind pattern. Windblown loess sediments generally are deposited in homogeneous blankets that can cover the countryside and are best preserved on flat topographic surfaces and the leeward side of hill slopes.

By the nature of its grain size, dune sand is relatively well sorted and clean. The sand grains are generally noncohesive but packed in an orderly arrangement with abundant point-to-point contacts. As such, dune sand locations in Colorado [dune coverage from Madole and others (2005) is shown in **figure 5-14**] are rarely susceptible to hydrocompaction when wetted. Older dune deposits are even less susceptible because pedogenic, illuvial clay has further bound or cemented the grain contacts as soil horizons begin to develop.

Loess, composed predominantly of silt and clay with lesser amounts of fine-grained sand, tends to be deposited as very dry dust. Where clay particles bridge the larger silt and sand grains, the resulting tenuous skeletal contacts create the collapse-prone low-density, metastable, soil fabric that was discussed in Chapter 3. The index properties of loess are dependent on the source materials, which can range from low-plasticity silt that is hydrocompactive to predominantly smectitic silty clay with swelling-clay characteristics.

Worldwide, most hydrocompaction has occurred in loess soils, and there are many references on this topic. A major symposium emphasizing loess soils, the NATO Advanced Research Workshop on Genesis and Properties of Collapsible Soils, was held in the United Kingdom (Derbyshire and others, 1994). Rogers and others (1994) reviewed studies from China, Russia, North America, and Europe. The literature records occurrences in Israel (Zur and Wiseman, 1973), Libya (Anagnosti, 1973), and eastern European areas such as Poland (Palka and Naborczyk, 1985; Grabowska-Olszewska, 1988), Bulgaria (Minkov and others, 1985), and the former nations of Czechoslovakia (Feda, 1966) and Yugoslavia (Milovic, 1988). In Asia, loess is found in Thailand, in addition to the huge expanse of loess deposits in China (Phien-wej and others, 1991). Major U.S. loess deposits occur in the Midwest and in dry upland areas of the Columbia Plateau in southwest Idaho and eastern Washington and Oregon.

The majority of Colorado eolian deposits are in the Great Plains Province (fig. 5-14). Madole (1995) analyzed soils derived from eolian sediments and compiled coverages based on NRCS soil series descriptions. He stated that 60 percent of Colorado east of the Rocky Mountains is covered in windblown deposits; about 30 percent is sand, and the remaining 70 percent is loess. Eolian sediments approach 200 ft in thickness in certain areas of eastern Colorado. Arroyos and tributaries of the Arikaree River near the Beecher Island battlefield expose some of the thickest eolian deposits, predominantly loess, in Colorado (**figure 5-15**). There are no other significant dune sand deposits in central or western Colorado except for the Great Sand Dunes National Monument in the San Luis Valley and some smaller dunes near Walden in North Park. There are scattered small, but significant, loess deposits in the west-central river valleys and the Wyoming Basin and Colorado Plateau physiographic provinces.

Loess soils are responsible for most of the hydrocompactive settlement problems along the urban corridor of the Colorado Front Range. In western Colorado, because of their age and relative thinness, loess deposits generally do not have a high potential for collapse upon wetting, unless they have been reworked and laid as a more recent soil deposit or if they contain large concentrations of pedogenic gypsum.

GYPSIFEROUS SOILS AND SOILS DERIVED FROM EVAPORITE BEDROCK

Gypsiferous soils occur in semiarid terrains of Colorado and can be either authigenic (formed in place, also termed "pedogenic") or allogenic (transported). Authigenic gypsum is chemically precipitated and is present as dispersed grains or crystals as a soil-cementation agent; clast coatings, filaments, or

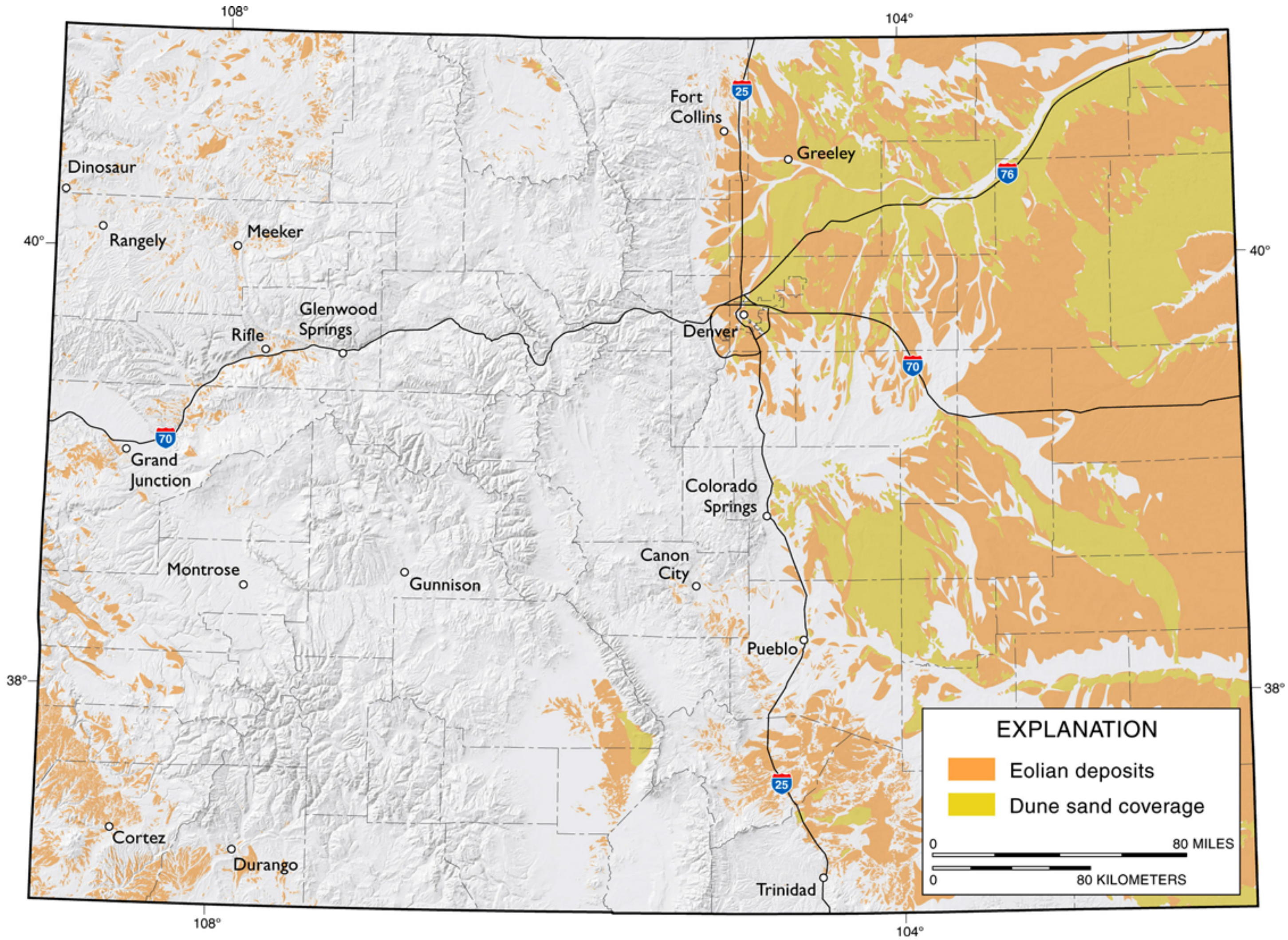


Figure 5-14. Areas of Colorado with Holocene and Pleistocene surficial eolian deposits. Compiled from NRCS SSURGO data and Madole and others (2005).

Figure 5-15. Thick loess exposed along the tributary of Black Wolf Creek near the confluence with the Arikaree River, 2 mi north of Beecher Island Historical Monument in northeastern Colorado.

nodules; or, ultimately, as a more massive accumulation of an easily identifiable, gypsiferous soil horizon that is part of the soil-formation process (pedogenesis). Allogenic gypsiferous soils are formed when gypsiferous bedrock is eroded and sediment with a high concentration of discrete gypsum grains is deposited and subsequently forms soil. Both types of gypsiferous soils can be collapsible.

Authigenic gypsiferous, silty-clay soils are commonly found in alluvium and mud flats derived from Mancos Shale in western Colorado, specifically in the Rangely area; the Grand Valley; the Uncompahgre Valley near Montrose; southeast of Trinidad; and in loess caps on low mesas in the Four Corners region in the Ute Mountain Indian Reservation, south of Cortez. NRCS soil scientists at the Cortez NRCS field office have reported soil horizons that are as much as 40 percent pedogenic gypsum (Doug Ramsey, personal commun., 2004).

Allogenic gypsiferous soils are derived from exposed evaporite rock, which is composed of predominantly evaporite minerals and fine-grained clastic rocks. The leaching of dissolved minerals from exposed evaporite rock can leave a thin crust of residuum that erodes easily and becomes transported sediments in alluvial fans and colluvial slope wash. Evaporite rock formations in several areas of Colorado (**figure 5-16**) have been mined for gypsum historically to make plaster of paris and more recently to make sheet rock for the construction industry. Evaporite rock is composed of the common evaporite minerals gypsum ($CaSO_4 \bullet 2H_2O$), anhydrite ($CaSO_4$), and halite (rock salt—NaCl), which are usually associated with thinly interbedded fine-grained sandstone, mudstone, and black shale layers. These evaporite minerals precipitated from seawater during cyclical evaporation of shallow seas and formed thick deposits. Millions of years of burial, plastic deformation, mountain building, and erosion have exposed the evaporite bedrock at or near the present ground surface.

Evaporite minerals easily dissolve in the presence of fresh water. Rock salt is so soluble it will dissolve before ever being exposed at the surface, even in the semiarid climates of Colorado. Gypsum, the more common mineral seen at the surface, is five times more soluble than limestone (Brune, 1965). It is this dissolution of the evaporite rock that creates caverns, open fissures, depressions, breccia pipes, subsidence troughs, and sinkholes—landforms that exemplify karst morphology. Classic karst-subsidence collapse features, although a hazard in many parts of Colorado, are not addressed in this publication.

Evaporite rocks are pertinent to the research of collapsible soils because of the soil-development processes and the soil types derived from these rocks. In populated areas of west-central Colorado, evaporite bedrock is present within the drainage basins and is the sediment source of alluvial fans and colluvial slopes. Soils in these depositional settings have high percentages of gypsum, as both allogenic detrital grains and authigenic precipitates, which can form soil

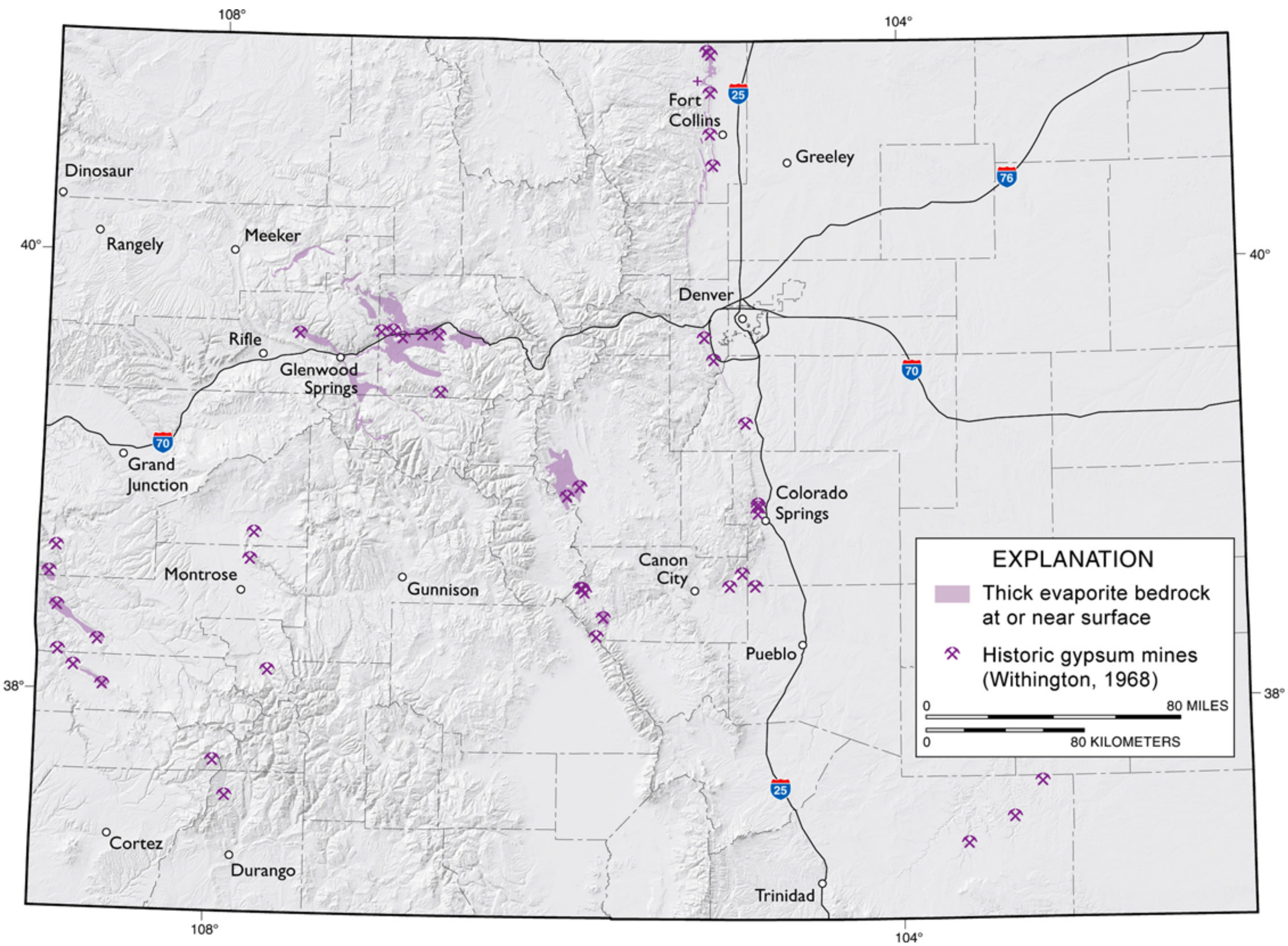


Figure 5-16. Areas of Colorado with thick evaporite bedrock near or at surface and sites of historic gypsum mining.

particle–binding agents. The soft evaporite rock easily erodes to fine-grained sediments (fig. 5-3). Soils from such sediments are generally even more susceptible to collapse upon wetting than nongypsiferous soils in similar depositional systems. Fine-grained alluvial-fan and colluvial soils derived from evaporite bedrock have been found with dry densities as low at 70 pcf, natural moisture as low as 3 percent, and collapse potential greater than 10 percent in consolidation testing at 1,000 psf loads (White, 1998). Similar occurrences of high collapse potential in sediments derived from evaporite rocks have been reported in the semi-arid climates of Spain (Salas and others, 1973; Gutiérrez and others, 2000).

NEAR-SURFACE WEATHERING AND GYPSUM RECRYSTALLIZATION OF MANCOS SHALE

The Mancos Shale is being actively investigated by the USGS in the Gunnison Gorge National Conservation Area as part of their collaborative research with other government agencies in Cretaceous black shale terranes. The primary focus of this study is to understand the weathering and erosion of the shale and the salt and selenium content thereof. Subsurface investigations by the USGS reveal that the weathered zone of the Mancos Shale can extend to a 40-ft depth (Richard Grauch, oral commun., 2006). In areas of the Grand and Uncompahgre valleys, from the Utah border to Montrose, the Mancos Shale is characterized by near-surface weathered zones where fractures in claystone have been infilled with crystalline gypsum (**figure 5-17**). This authigenic gypsum is thought to be formed by a geochemical-microbiological oxidation weathering process in the shale. The Mancos Shale was deposited in a deep-water, reduced environment and contains significant amounts of disseminated pyrite. Within the weathered zone, oxidation of pyrite creates sulfuric acid that reacts with available calcite in the shale to form gypsum. The gypsum tends to crystallize along bedding and fracture planes in the shale. The volume change when calcite recrystallizes to gypsum and during the further growth of gypsum crystals can heave the claystone, physically fracturing, splitting, and jacking it apart. The heave from such expansion of pyritic shale creates significant problems in many parts of the United States and Canada (Penner and others, 2002). Subsequent wetting of this weathered "jacked" claystone can result in the dissolution of the gypsum, micropiping erosion, and creation of subsurface voids.

In the Mancos Shale adobe hills (fig. 5-3A) and within the buff to tan, weathered zones below mesa gravel caps, the fabric of the claystone has been mechanically expanded. The weathered claystone has been wedged apart and split horizontally along bedding planes and randomly along fractures. These miniature horizontal fissures are open or partially filled with recrystallized gypsum (**figure 5-18**). This condition can result in a "collapsible Mancos claystone," and foundations on this type of formational claystone may settle upon loading. Although Mancos Shale is usually expansive, in this case, long-term wetting dissolves and softens the gypsum, resulting in a recompression of the split and fractured, weathered, formational claystone when loaded. This behavior can be a significant problem in collapsible-soil areas where driven piles or caissons rely predominantly on end-bearing loading. Negative skin friction from the collapsing soil column above may result in punch-through of the pile tip into the recompressing Mancos Shale, which could manifest itself as foundation settlement.

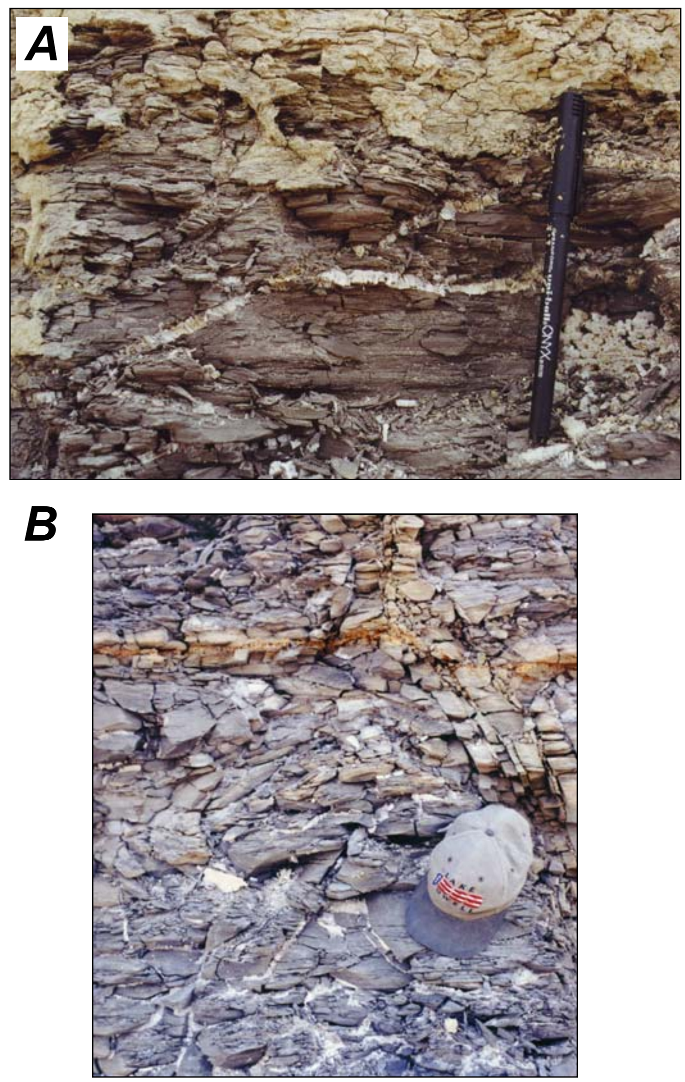


Figure 5-17. Gypsum fracture filling (white) in Mancos Shale. *A*, Shale exposure on flank of Flat Top Mesa near Montrose (pen for scale). *B*, Near the east dam abutment of Highline Reservoir north of Mack (baseball cap for scale).

Figure 5-18. Weathered Mancos Shale with bedding-plane fractures that are partially filled with gypsum (2-in.-wide bottle cap for scale). Specimen courtesy of E. Morris, Grand Junction Lincoln-DeVore.

Soil scientists refer to this leached and weathered claystone horizon as the “Blond” Mancos (David Dearstyne, personal commun., 2004). There are case histories in the Grand Junction area north of Interstate 70 and south of the Highline Irrigation Canal where Mancos claystone settlement has reportedly caused structural problems in both light and heavily loaded residential and commercial structures (Ed Morris, personal commun., 1997). The Highline Dam at Highline Lake State Park had 3.5 ft of embankment settlement, a part of which was attributed to piping and recompression of weathered claystone bedrock (Bruce Marvin, personal commun., 1998). The Colorado Division of Wildlife, the dam owner, was required to spend $1.12 million in remedial work for repair, refurbishment, and continued monitoring. In a recent development application in Mesa County near Whitewater, south of Grand Junction, the geotechnical consultant reported the results of a consolidation test of a sample of weathered Mancos claystone that collapsed 6 percent at a surcharge load of 1,000 psf, which is a significant and unusual collapse for formational claystone.

6. PUBLISHED RECORD OF COLLAPSIBLE SOILS

NATIONAL AND INTERNATIONAL OVERVIEW

In the 1950s through the 1970s, a number of pioneering reports were published that discussed the physical characteristics, engineering properties, and collapse mechanisms of various soils from different geomorphologic settings (Jennings and Knight, 1957, 1975; Clevenger, 1958; Lofgren, 1960, 1969; Holtz and Hilf, 1961; Gibbs and Bara, 1962, 1967; Knight, 1963; Bull, 1964; Feda, 1966; Sultan, 1969; Dudley, 1970; Barden and others, 1973). These reports laid the foundation for more recent studies of the occurrences, identification, and proper geotechnical engineering of collapsible soils. The CGS formally defined the phenomenon in Colorado in 1974 (Rogers and others, 1974). Since 1995, there has been much research worldwide on collapsible soils. In Europe, NATO sponsored the Advanced Research Workshop on Genesis and Properties of Collapsible Soils in 1995 (Derbyshire and others, 1994), and the International Association of Engineering Geologists (IAEG) created a Commission on Collapsible Soils (no. C18) in 1999. The American Society of Civil Engineers (ASCE) Geo-Institute has a subcommittee on unsaturated soils and sponsors sessions at the Geo-Institute biennual meetings (e.g., Houston and Fredlund, 1997) where collapsible soils are a major topic. Proceedings of these workshops and sessions have been published, and many of those reports have been reviewed and referenced in this publication.

There is also a body of more recent work on collapsible soils of the Western United States that is cited in this publication. Prominent research has been conducted since 1980 on engineering properties and mitigation methods by Sandra Houston of the Civil Engineering Department of Arizona State University. Characterization, mapping, and mitigation research has been conducted by Kyle M. Rollins of the Civil Engineering Department of Brigham Young University. Important case-history reports have been published by engineering consultants who have experience in collapse-prone terrain (e.g., Mock and Pawlak, 1983; Hepworth and Langfelder, 1988; Beckwith and Hansen, 1989; Hansen and others, 1989). Geological surveys of various Western States have studied collapsible-soil occurrences. Hazard maps have been developed for parts of Utah (Owens and Rollins, 1990; Williams and Rollins, 1991; Mulvey, 1992; Rollins and others, 1992a). In 1984, soil collapse affected the town of Il Llano near Espanola, New Mexico, so badly that the Governor declared a state of emergency (Shaw and Johnpeer, 1985a, 1985b). This incident drove further research in New Mexico by Reimers (1986).

College textbooks that discuss the urban geomorphology of dry lands and desert geomorphology also make passing comments on the formation of metastable soils in these types of environments (Cooke and others, 1982, 1993). It was the understanding that similar geology and geomorphology existed in Colorado—along with an increasing number of Colorado-specific distress and damage cases attributed to collapsible soils since the 1970s—that initiated the Colorado Statewide Collapsible Soil Study, as well as the recent regional studies and susceptibility mapping by the CGS.

COLORADO-SPECIFIC PUBLISHED RECORD

The published record of collapsible soils specific to Colorado, in terms of both classic hydrocompactive soils and piping and dispersive soils, extends back to the early 1900s. Most of the work occurred during the post–World War II surge in construction, the energy booms of the 1970s and ’80s, and recent growth in residential construction. The following is a synopsis of many of those published reports. **Figure 6-1** shows the locations referenced in these published citations.

The first published account of collapsible soils that was specific to Colorado was an early text on farming practices in the semiarid Western United States, specifically the planting of fruit orchards and irrigation of dry soils that had never been exposed to saturation or high ground water. Farming development in the Grand Valley began with the completion of the first irrigation canals in 1895 (Spears and Kleven, 1978). Ground problems occurred shortly thereafter. Although the process was not understood, fruit growers and agriculturists began to recognize the hazards of “sinking ground.” Early horticulturists (Paddock and Whipple, 1910) made one of the first references to collapsible soil:

> **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 15 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.

This description of the subsidence, downwarping, and formation of ground cracks upon wetting is similar to many published accounts of the results of ground wetting and soil saturation in collapsible soils research, including experiences in Colorado by CDOT along the Interstate 70 corridor west of Rifle (R.K. [Bob] Barrett, personal commun., 2004). Also, the observation that the soils have a porous condition is one of the diagnostics used today to determine collapse potential.

Paddock and Whipple (1910) also discussed preparation of land for planting of fruit orchards:

> In the arid regions there are types of land . . . that settle from one to three feet when irrigation water is applied; such areas should be thoroughly watered before an attempt is made to level them. As a rule the higher knolls settle

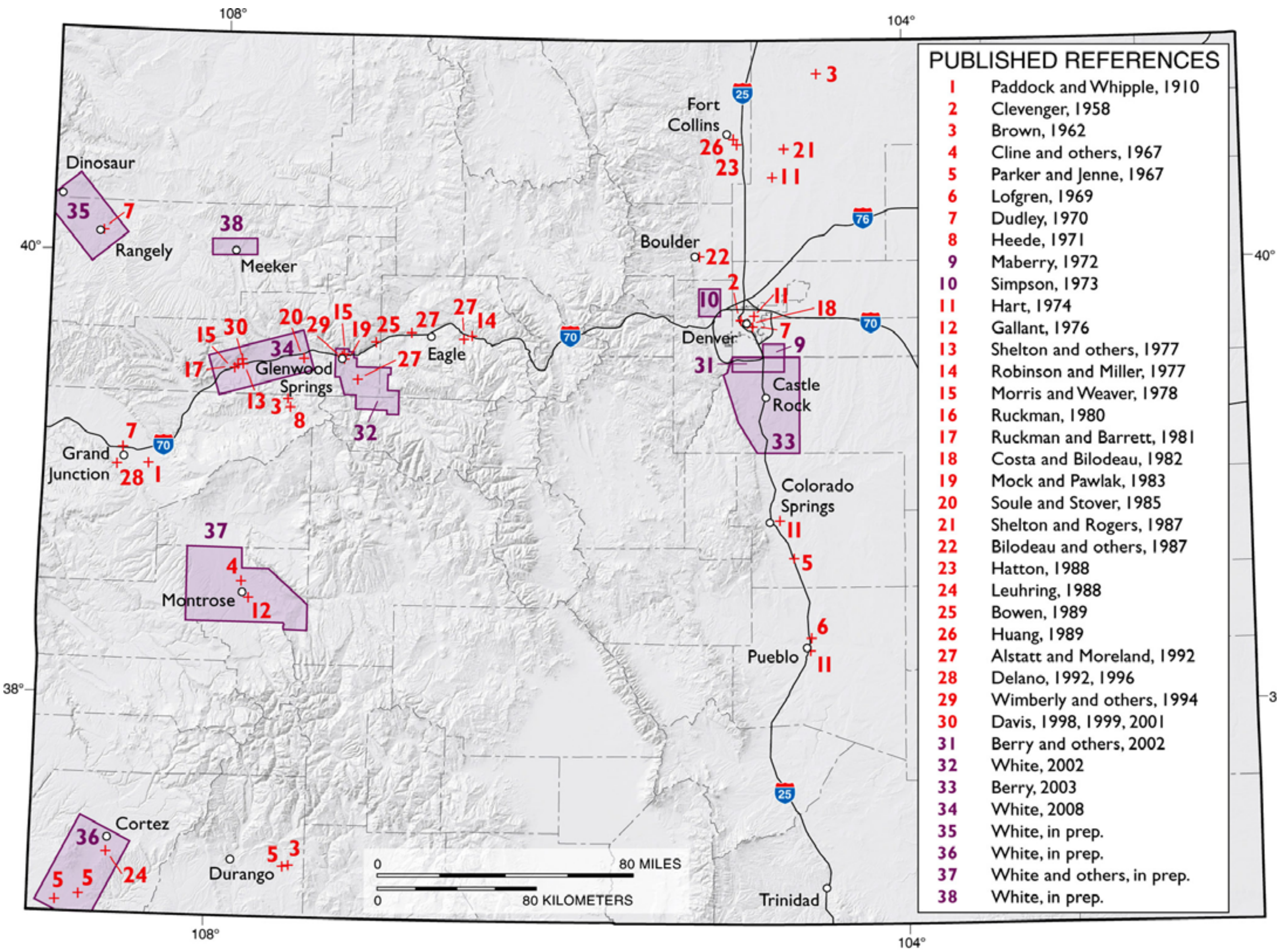


Figure 6-1. Locations of published references of collapsible soils in Colorado.

most, and leveling before settling often moves soil that must be moved back after the process of settling is completed.

This is one of the first recommendations for mitigation of collapsible-soil hazards by prewetting: when establishing new land for fruit orchards, fields should be flood irrigated for a suitable time to induce soil collapse, *before* final grading of the orchard field, excavation of irrigation channels, and planting of the fruit tree seedlings.

One of the first discussions of collapsible soils and settlement damage to residential foundations in Colorado was by Clevenger (1958). In two case histories, the affected buildings were constructed in the post–World War II, early 1950s expansion along the eastern to southeastern margin of the Denver city limits, in the Belcaro and Congress Park neighborhoods. The soils were derived from windblown loess that caps the heights above and to the east of the modern alluvium of the South Platte River and Cherry Creek valleys. Damage was attributed to wetting of the dry soils by lawn watering, broken water pipes, and proximity to the drain fields of septic systems. In Clevenger's analysis of Denver loess compared to loess in the Missouri River Basin area, the Denver loess was found to have higher smectite content, was better graded, and had higher dry density. However, settlement and loss of shear strength in the Denver loess was more "unfavorable" (in the sense that the collapse is greater), even though the soils were of higher density.

In special loading tests on loess deposits near Pueblo, Colorado, plate load tests on 2-ft-square and 4-ft-square pads were loaded with 2,500 lb and 3,340 lb, respectively (Lofgren, 1969). Upon flooding, the collapse settlement in the loess became so great that the load platforms toppled and the tests had to be stopped. Lofgren (1969) attributed the Pueblo reference to Clevenger (1958), but a reprint of that article as ASCE Transactions Paper no. 2916 does not contain the Pueblo references. More recent work on collapse of remolded eolian soils has been done by graduate students of John Nelson of Colorado State University. Hatton (1988) and Huang (1989) investigated certain collapse-prone loess soils outside of Fort Collins for their master's theses in the Civil Engineering Department.

Piping-erosion and soil-collapse voids that form pseudokarst morphology in Colorado were first discussed in a short report by Brown (1962), who examined piping soil locations in Bayfield in southwest Colorado, outside of Nunn in the Pawnee National Grasslands in northeast Colorado, and the watershed of Alkali Creek south of Silt in west-central Colorado. Parker and Jenne (1967) discussed piping hazards and damage to roadways in semiarid to arid climates in recently deposited (Holocene) fine-grained valley alluvium that has been entrenched by arroyos in the arid to semiarid southwest United States. Three Colorado-specific piping-erosion case histories in clay-rich alluvium were included: along U.S. 85 south of Colorado Springs; in southwestern Colorado near the town of Bayfield; and in certain arroyos that CO 140 crosses near the Four Corners region (fig. 3-5). The significance of soil dispersion and resultant piping and void formation in soils of the arroyos in Delta and Montrose Counties was mentioned in the *Soil Survey of Delta-Montrose Area, Colorado* (fig. 4-15) (Cline and others, 1967). Piping soils in west-central Colorado were further investigated by Heede (1971). Near the area of Brown's (1962) earlier work, Heede further described the Alkali Creek alluvium south of Silt and surveyed some of the larger piping tunnels. The soils were found to have high SAR values, which showed them to be very dispersive when plotted on the Sherard and others (1976b) susceptibility chart (fig. 4-17).

Large piping-type caves in unconsolidated alluvial soils in western Colorado have been explored and documented by members of the Colorado Grotto section of the National Speleological Society. A large "mud cave" in Holocene debris-flow and alluvial-fan deposits is present in the Anvil Points area, inside the former Naval Oil Shale Reserve north of Rulison; that cave has more than 2,000 ft of surveyed passages Davis (1998, 1999, 2001). A similar but smaller cave has been mentioned near the Colorado National Monument west of Grand Junction. This cave captured significant drainage, and the cave stream has partially incised weak mudstones of the Morrison Formation (Delano, 1992, 1996; Davis, 1992).

Dudley (1970), in his widely cited review of collapsible soils, mapped three instances of collapsible soils not formed from loess in the Denver, Rangely, and Grand Junction regions of Colorado. Regrettably, there was no specific discussion in the text about these Colorado locations.

In the early 1970s the U.S. Geological Survey (USGS) produced quadrangle maps at 1:24,000 scale of the greater Denver metro area that covered geology, soils, geography, and geologic hazards, including maps of surficial deposits (soils) that are susceptible to compaction or subsidence. The susceptible soils included both hydrocompactive soils and organic soils (the latter are not included in the discussion of this publication). These maps are available for the Parker Quadrangle in Arapahoe and Douglas Counties (Maberry, 1972) and the Golden Quadrangle in Jefferson County (Simpson, 1973).

During the energy boom of the 1970s, when Interstate 70 was under construction, the hydrocompactive properties of soils along that alignment west of Rifle were investigated by CDOT and CGS (Shelton and others, 1977). On certain alluvial fans, collapsible soils as thick as 90 ft were identified by CDOT along the Interstate 70 alignment (Ruckman, 1980; Ruckman and Barrett, 1981). In 1978, at an ASCE-sponsored conference on collapsible soils, Ruckman stated that, prior to construction, prewetting mitigation techniques were used for 15 mi of highway alignment (**figure 6-2**) and prewetting another 35-mi stretch had been proposed for the future, with an anticipated cost of $60,000 per mile. During construction of Interstate 70 through Glenwood Canyon, a settling bridge abutment in hydrocompactive soils on a small alluvial fan at the Bair Ranch Rest Area required compaction-grouting remedial repair (Bowen, 1989):

As part of the popular ongoing series of the geology of major U.S. cities in the Association of Engineering Geologists quarterly bulletins, Costa and Bilodeau (1982) and Bilodeau and others (1987) discussed collapse-prone soils in Denver and Boulder, mentioning soils composed of loess that have been known to be susceptible to soil collapse when wetted. The locations noted in the Denver report are the same as was previously cited by Clevenger (1958).

Extensive settlement damage to existing structures was also documented in the intermontane and Colorado Plateau Province valleys of the Eagle, White, Roaring Fork, and Colorado Rivers, where the valley floors have a semiarid climate. Several investigations included an in-depth discussion and map of hydrocompactive soil hazards in Glenwood Springs (Morris and Weaver, 1978). Mock and Pawlak (1983) were the first in Colorado to analyze case histories (45 sites) of damage to residential structures due to the wetting and collapse of hydrocompactive soils on alluvial fans and colluvial

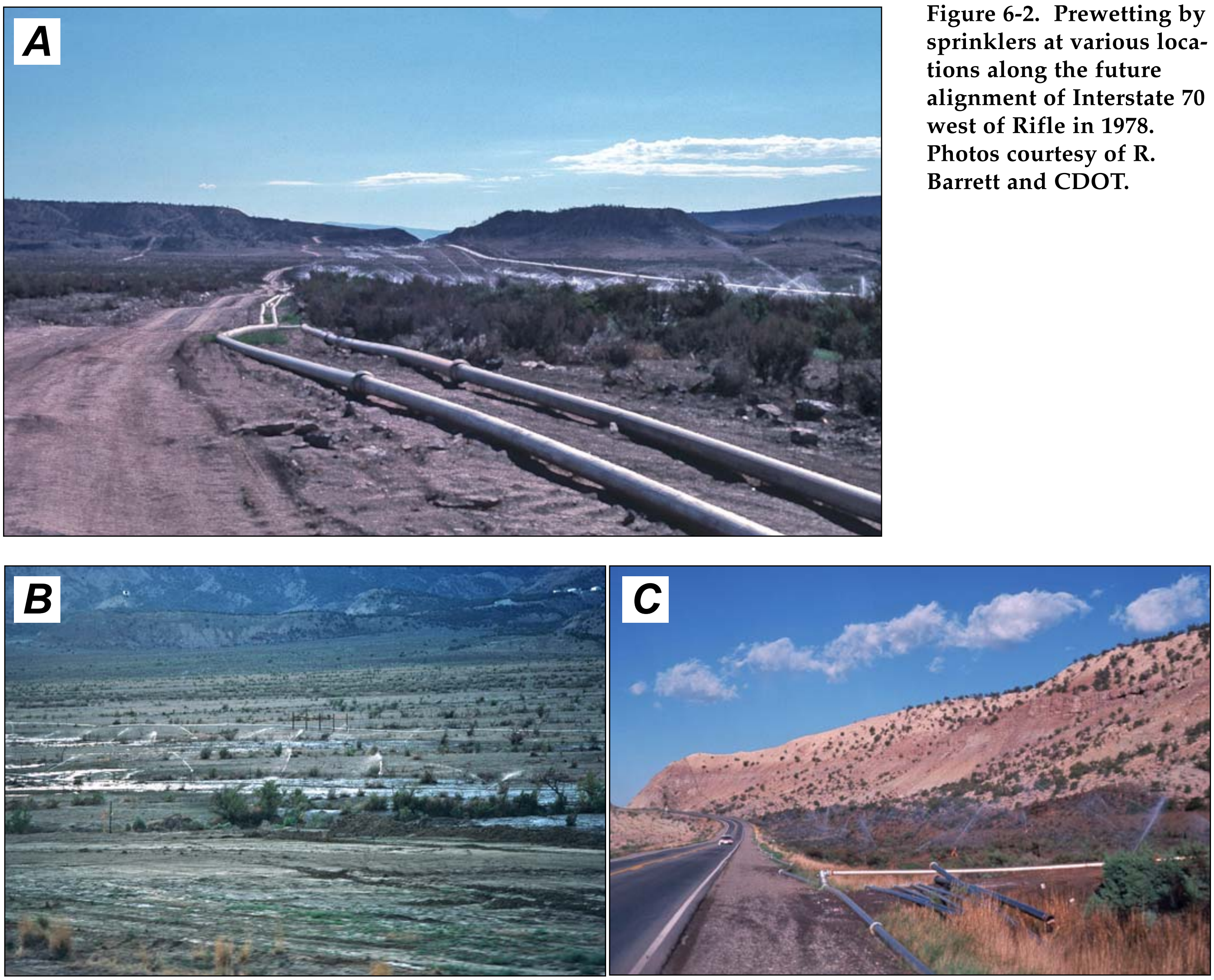


Figure 6-2. Prewetting by sprinklers at various locations along the future alignment of Interstate 70 west of Rifle in 1978. Photos courtesy of R. Barrett and CDOT.

hillside slopes, which commonly mantle the Roaring Fork valley bottom near the confluence with the Colorado River in Glenwood Springs. From their work in cataloging soil-test data acquired from their practice in foundation and geotechnical engineering, they developed the susceptibility chart that was discussed in Chapter 4. Wimberly and others (1994) investigated the Glenwood Springs water treatment plant, which experienced significant damage when leakage occurred from failure and settlement of the underlying rocky, hydrocompactive, colluvial soils. Their paper described the investigation, soil conditions, and the compaction-grouting that stabilized the structure.

The U.S. Bureau of Reclamation (USBR) has had much experience in hydrocompactive soils, predominantly as part of the extensive aqueduct projects serving the water needs of California (Gibbs and Bara, 1962; Curtin, 1973; Poland, 1973; Prokopovich, 1984). Similar problems were noted in irrigation canals and infrastructure in the extremely dry Montezuma Valley, where broad alluvial fans and mud flats blanket the ground surface south of Cortez along U. S. Highway 160, near the Ute Mountain Reservation town of Towaoc. An exceptionally detailed work on collapsible-soil genesis, evaluation, mitigation methods, and the evolution of engineering methods to address such soils was completed as a master's thesis in civil engineering by Luehring (1988), who studied collapse susceptibility in alluvial-fan deposits at the Towaoc Canal as a geotechnical engineer with the USBR.

The CGS has recognized the risks and hazards for development on collapsible soils since the early 1970s. The CGS publication *Guidelines and Criteria for Identification and Land-Use Controls of Geologic Hazards and Mineral Resource Areas* (Rogers and others, 1974) includes the descriptive definition:

> *Hydrocompaction produces ground surface collapse from excessive wetting of certain low-density weak soils*. This can occur in two general types of soil that are common in Colorado: a) wind deposited soils (loess), and b) predominantly fine-grained colluvial soils. In either case, collapse occurs from excessive wetting of previously dry, collapsible soils. Wetting of these materials weakens the already weak or unstable soil structure, which undergoes internal collapse and densification (reduction of air voids). Densification of the weak soil column produces ground surface collapse and subsidence in the vicinity of excessive wetting. Removal of fine material by piping is probably an additional factor in some cases of subsidence by wetting. Such excessive wetting can occur from irrigation, broken water lines, surface ponding, or drainage diversions.
>
> Wind-blown silt (loess) deposits cover broad areas of Colorado from the Front Range to the eastern border of the state. Predominantly fine-grained colluvial soils are generally associated with mountainous areas where they occur as moderately sloping surfaces (colluvial wedges) between steep valley sides and deposits of the valley floor.

The quoted description omits the identification of alluvial fans and recent fine-grained alluvium that are possibly more prone to hydrocompaction and piping than loess deposits and colluvium soils in Colorado, on the basis of more recent research and occurrences.

Ground settlement was included as a defined geologic hazard in House Bill 1041, which was enacted in 1974 [State statute 24-65.1-103 (10), Colorado Revised Statutes, 2007]. Part of House Bill 1041 charged local governments with identifying geologic hazards and implementing guidelines for appropriate land use within their jurisdictions. Monies were made available for counties to map geologic hazards for critical growth areas. Some county maps specifically show areas of subsidence potential related to hydrocompactive soils (e.g., Gallant, 1976; Robinson and Miller, 1977). CGS is currently cataloging these maps into a searchable database for the CGS web page. The House Bill 1041 maps can be found at some county planning offices or may be reproduced from CGS files, upon request.

At the same time as House Bill 1041 passed, the CGS published a series of maps of potentially swelling soil along the Front Range (Hart, 1974). These maps described windblown sand and silt as being subject to severe settlement or hydrocompaction.

There are several other CGS publications on collapsible soils. Shelton and Prouty (1979) presented brief case histories of collapsible soils located in Boulder, the Roaring Fork Valley, and the highway construction of Interstate 70 in western Colorado. A geologic hazards map of Garfield County at a scale of 1:50,000 in Soule and Stover (1985) specifically includes hydrocompactive soil. On this map, active alluvial fans that were designated only as debris-flow hazards also typically contained collapsible soils, but were not labeled as such because map units were annotated by a single hazard. Mapping of the Windsor area of Larimer and Weld Counties showed shallow surficial materials composed of wind-blown sediments that may be prone to settlement when saturated and loaded (Shelton and Rogers, 1987). In addition to publications, proceedings from CGS-sponsored geologic hazards conferences included specific reports on hydrocompactive soils and locations in the State where they are likely to be encountered (Olsen, 1996; White, 1998, 2001; Pawlak, 1998).

Regional maps specific to collapsible soils are also being completed by CGS. White (2002) developed a collapsible-soil susceptibility map of the Roaring Fork Corridor from Basalt to Glenwood Springs, based on geomorphology, geologic source areas of sediments, and climate. This 1:50,000-scale derivative map used new CGS 1:24,000-scale geologic mapping that emphasized surficial deposits, NRCS soil-survey data, and a compilation of case histories that were generated as part of the statewide collapsible-soils database established for this publication. The White (2002) map also included a block diagram using a 10-m digital elevation model so that the three-dimensional geomorphology of the deposits could be better shown (reproduced in fig. 5-5B). The other regional study completed in 2002 was the Douglas County study (Berry and others, 2002). This series of maps of the Highlands Ranch area in the southern Denver metro area delineated swelling soils and collapse-prone soils by analyzing compiled soil-test data (including soil-index tests and swell/consolidation results from consultants' reports) and NRCS soil-survey data of mapped soil units and their soil-index properties. This information was discussed in Chapter 4. NRCS soil-survey data were also used to delineate the potential for soil collapse for the entire Douglas County (Berry, 2003). These data—in geographic information system (GIS) Environmental Systems Research Institute, Inc. (ESRI) shapefiles—are available from the GIS data link at the CGS Web site.

Since the mid-1990s the CGS and USGS have undertaken geologic mapping at the 1:24,000 quadrangle scale in locations of Colorado where there is development growth pressure. The maps show recent surficial deposits (engineering soils) and generally contain a discussion on potential geologic hazards inherent to each deposit type. Hydrocompactive and piping hazards are specifically mentioned in the text describing the map units along the Colorado River and Eagle River corridors, as well as in maps of areas along the lower Roaring Fork Corridor referenced in White (2002). The geologic map comments are brief, and a complete description of the morphology of collapsible soils and the geomorphic systems in which they form is lacking. More comprehensive derivative mapping of collapsible-soil susceptibility in those areas, as well as others shown figure 6.1, have been completed and are being prepared for publication. Currently available is another 1:50,000-scale map that details the collapsible-soil susceptibility of the Rifle area (White, 2008) in Garfield County, and a susceptibility map of the Uncompahgre River Valley area mapped at a 1:24,000-scale, which is one of a series of geologic hazard maps developed for Montrose County that are available as GIS ESRI shapefiles and 1:50,000-scale Adobe (pdf) image files (White and others, in prep).

The NRCS has studied collapsible gypsiferous soils statewide. Nettleton and others (1982) reported authigenic (i.e., formed in place, pedogenic) gypsiferous soils on the eastern plains in eastern Arapahoe, Morgan, and Prowers Counties, in addition to Alamosa County in the San Luis Valley. As mentioned earlier in this text, NRCS soil mapping in the Montezuma Valley at the Ute Mountain Indian Reservation has also revealed soils with heavy gypsic horizons (Doug Ramsey, personal commun., 2004). Alstatt and Moreland (1992) mapped and described prominent allogenic (i.e., transported) soils derived from the evaporite bedrock of Eagle and Garfield Counties. These types of soils are unstable and poorly suited for home-site development; the main limitations are erosion, piping, and low soil strength during wet periods (Alstatt and Moreland, 1992; Nettleton and others, 1982). Prewetting foundation subgrades was discussed as a mitigation technique for residential construction.

7. COLORADO GEOMORPHIC REGIONS SUSCEPTIBLE TO COLLAPSIBLE SOILS

This chapter discusses the physiographic provinces in Colorado where geomorphic systems associated with collapsible soils could occur; the chapter also presents case histories from the statewide database. The state of Colorado has three basic physiographic provinces, the Great Plains of eastern Colorado, the central Rocky Mountains (more formally, the Southern Rocky Mountain province), and the western Colorado Plateau region (Fenneman and Johnson, 1946). These provinces are shown in figure 5-2 and plate 1. Occurrences of collapsible soils within each of these provinces are discussed in this chapter.

GREAT PLAINS PROVINCE

The Great Plains physiographic province encompasses the eastern third of Colorado. The western border of the province, from the Wyoming border to Colorado Springs, consists of the mountain front termed "the Front Range" and its associated foothills. South of Colorado Springs, the demarcation is less linear and less distinct, undulating westward at the Cañon City Embayment and eastward around the Wet Mountains before following the eastern flank of the Sangre de Cristo Mountains to New Mexico.

The Great Plains Province includes the drainage basins of the South Platte River and the Arkansas River, both of which originate in the Rocky Mountains. The Palmer Divide near Monument separates the drainage basin of the South Platte River that flows northeast from that of the Arkansas River that flows southeast. The Great Plains is subdivided into the High Plains to the east, the Raton Basin to the south, and the Colorado Piedmont in the west.

The Great Plains are considered dry lands; their average annual precipitation is only about 15 in. A notable exception is the Palmer Divide that includes elevations up to 7,600 ft in the western part of the province. The higher terrain of this major divide contains the Black Forest, the only forest of the eastern plains of Colorado, where annual precipitation can approach 20 in. Precipitation is lower over the south-central Colorado plains; the annual rainfall averages only about 12 in. from La Junta to Pueblo and the Cañon City Embayment.

Collapsible soils within the Great Plains are associated with three basic depositional processes: wind-blown or eolian sediments, geologically recent (i.e., Holocene) fine-grained alluvium in flood plains of tributary rivers and secondary streams, and slope-wash and hillside colluvial soils. Eolian sediments cover large parts of eastern Colorado (fig. 5-14), and the predominant collapsible-soil type in this area is loess.

High Plains

The High Plains terrain is typified by subdued, wind-swept hills and low escarpments of Tertiary rocks and by steeper slopes and bluffs along river valleys. Eolian deposition has produced some impressive landscapes, such as the Wray Sand Dunes. South of Wray some of the thickest loess sheets in Colorado are exposed along tributaries of the Arikaree River near Beecher Island (fig. 5-15). Although the loess is almost certainly hydrocompactive, there have been no documented incidents of collapse in this rural and agrarian area of eastern Colorado. There are many closed depressions on the loess sheets in eastern Colorado. Many of these ground depressions have the same northwest trend as the wind patterns and possibly originated as wind-scour landforms. However, a component of the depressions' formation may be subsidence and piping that occur when precipitation rates are high and water ponds and infiltrates into the loess. Colluvial slopes and flood plains within the larger creek valleys may also contain collapsible soils. The old church at the Beecher Island Battlefield site, on the current flood plain of the Arikaree, has cracked walls that might be attributed to hydrocompactive settlement of recent alluvium and slope colluvium derived from loess and Pierre Shale.

Raton Basin

The deposits prone to collapse in the Raton Basin include alluvial fans, colluvial slopes, slack-water deposits on present-day flood plains, and intermittent and possibly reworked loess deposits. The fine-grained soils in arroyos are also prone to piping erosion and the formation of pseudokarst landforms. A good example of piping and pseudokarst in loess and alluvial soils can be found just south of Walsenburg (**figure 7-1**). Depressions and sinkholes are present adjacent to a former pond and next to an arroyo. The NRCS soil survey described the soil as sandy loam that formed from eolian silt and fine-sand sediments; the sandy loam is distributed adjacent to silty-clay loam that was derived from colluvial slope wash (McCullough and others, 1983). The tendency for piping and shrink/swell behavior in these soils was also mentioned in the survey. Other examples of arroyo formation are in tributary alluvium of the

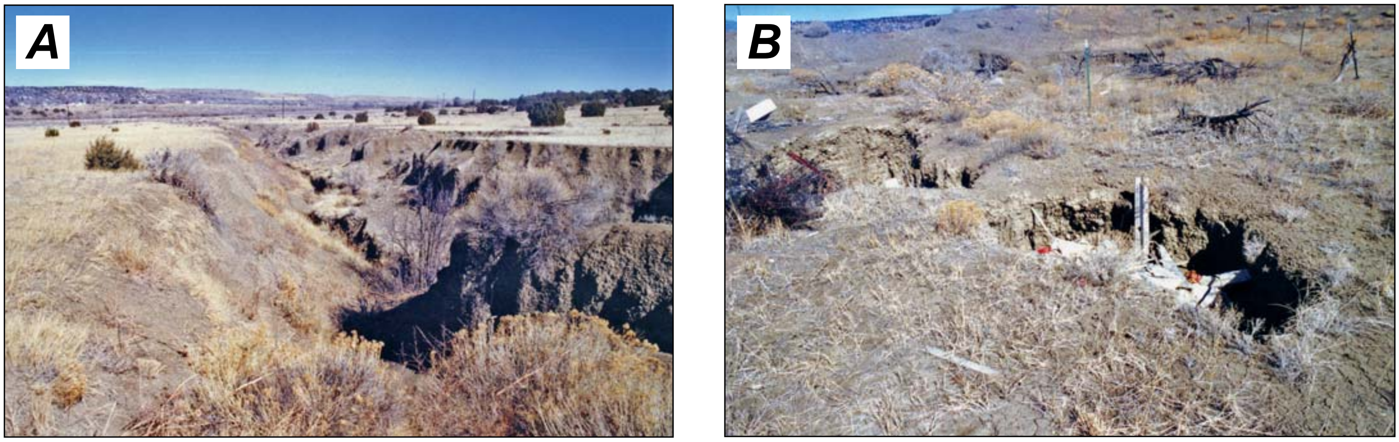


Figure 7-1A, Arroyo formation, piping, and pseudo-karst formation in clayey-silt soils near Walsenburg.

B and C, Trash has been dumped into the smaller sinkholes.

Purgatoire River in the Trinidad area. Overall, the climate, geomorphology, and deposition of sediments in the Raton Basin are similar to other areas of the State with collapse-prone soils, but the lack of development in the area has resulted in a paucity of case histories, leaving the extent and severity of collapsible soils in this area uncertain.

Colorado Piedmont

The Colorado Piedmont has been severely eroded by the many rivers that have exited canyons of the Front Range onto the plains; thus, the younger rock formations seen in the High Plains have been generally stripped away, exposing Cretaceous to Paleocene shale and sandstone. The geomorphology is characterized mostly by gently sloped pediments, dissected bluffs, benched river terraces alongside valleys, and subdued interfluvial hills. The major urban centers of the Front Range are located in the Colorado Piedmont, which explains why almost all of the collapsible-soil case histories for the Great Plains are found in this subprovince.

Analysis of the case histories reveals that eolian, recent slack-water alluvial, and hillside slope-wash colluvial depositional systems generate collapsible soils. Because of the complexity of these systems and the interrelationship with the more common swelling soils along the Front Range urban corridor, these three types of soil deposits are individually discussed here.

Piedmont Eolian Deposits

Collapsible eolian sediments are prevalent in the interfluvial hills that divide the many tributary rivers of the South Platte River from Fort Collins and Greeley to the Denver area, as well as in eastern Colorado Springs and the hills east of Fountain Creek, near Security and Widefield. As sediments were transported on the wind from the flood plains, the sands were deposited close to the drainages and the finer-grained silts settled farther to the east and southeast, following the predominant wind direction. This rough zonation is illustrated by Costa and Bilodeau (1982) for areas near the South Platte River and Cherry Creek. Loess deposits of varying thickness are present in eastern Denver, probably largely derived from the flood plain of Cherry Creek. Older brick homes have been affected, as roof gutters and underground pipes deteriorate, causing leakage and subsequent collapse below and around foundations and flat work (concrete slabs for driveways, garage floors, sidewalks) (**figure 7-2**). Loess also mantles the older pediment alluvial surfaces such as the Verdos and Slocum alluvial deposits in the southwestern Denver metro area and in the Pueblo area (Scott, 1964). Distress is noted where cracks form from settlement of the internal nonbearing walls, while the load-bearing walls on the more deeply founded footings and bearing pads have not moved (**figure 7-3**). Localized deposits of loess may be found on the leeward side of flood plains and commonly

Figure 7-2. Differential settlement of the back side of this home, possibly a later addition, has unhinged house and caused distress to the brickwork and roof. The underlying soil is loess.

cover, in varying thicknesses, the subdued hills of near-surface bedrock between flood-plain valleys (Shelton and Rogers, 1987; Madole, 1995). Caps of fine-grained Pleistocene alluvial sediments on topographic highs have also been the source of locally thick loess deposits found leeward of Dawson Formation exposures in Colorado Springs (at Venetucci and Cheyenne Mountain Boulevard) and east of Longmont (Paul Santi, personal commun., 2002). Near Greeley, runoff saturation and soil collapse of windblown silt beneath a U.S. Highway 34 bridge-approach fill embankment caused settlement damage to the structure (**figure 7-4**).

Where wind erodes alluvial deposits that are derived from shales and claystones or where there is

Figure 7-3. Crack in drywall (shown by arrows) formed by settling of the lower stairway wall in the foyer that is supported by a slab-on-grade foundation. The upper wall (above the crack), supported by floor joists that span to load-bearing walls at the foundation perimeter and mid-home-bearing beams, is not affected by the settlement.

Figure 7-4. Severe settlement of bridge-abutment approach embankment, from collapse of underlying loess soil, has damaged the retaining wall and cast-in-place guardrail structure. The abutment on the right side of the photograph, founded on deep piles that were advanced through the collapsible soil, is stable.

direct ablation of claystone residuum, windblown sediments are composed of clay-sized particles. A large percentage of eolian sediments that have been deposited along the Front Range have smectite clay (Clevenger, 1958). The resulting loess and colluvial slope-wash soils might exhibit both collapse and expansion. It is important to restate that sediment collapse is predominantly a mechanical phenomenon, whereas swell potential is based on the clay minerals present. In the Widefield section of southeastern Colorado Springs (which is adjacent to and underlain by Pierre Shale), thick sections of eolian sediment that can collapse by as much as 14 percent are mitigated by overexcavation before construction. However, when the material is remolded to be used as structural fill, the swell behavior is dominant and registers as low to moderate. The variable swelling-clay mineral content helps to explain the proximity of soils with collapse and swell properties, as identified by swell/consolidation testing compiled by Berry and others (2002) (**figure 7-5**). The juxtaposition of soils with such widely dissimilar properties poses challenging geotechnical engineering issues where both swell and collapsible-soil behavior may need to be addressed.

Sand and silt are common components of eolian deposits. Silty sand could be problematic for building, depending on the concentration of silt, clay content, and the thickness of the sand unit. In Colorado Springs, at International Circle where an educational facility was proposed, more than 25 ft of eolian silty sand are present; silt and clay compose only 15 percent of the sediment. Consolidation testing of the sand revealed 3 percent collapse at 1,000 psf. Similar conditions were

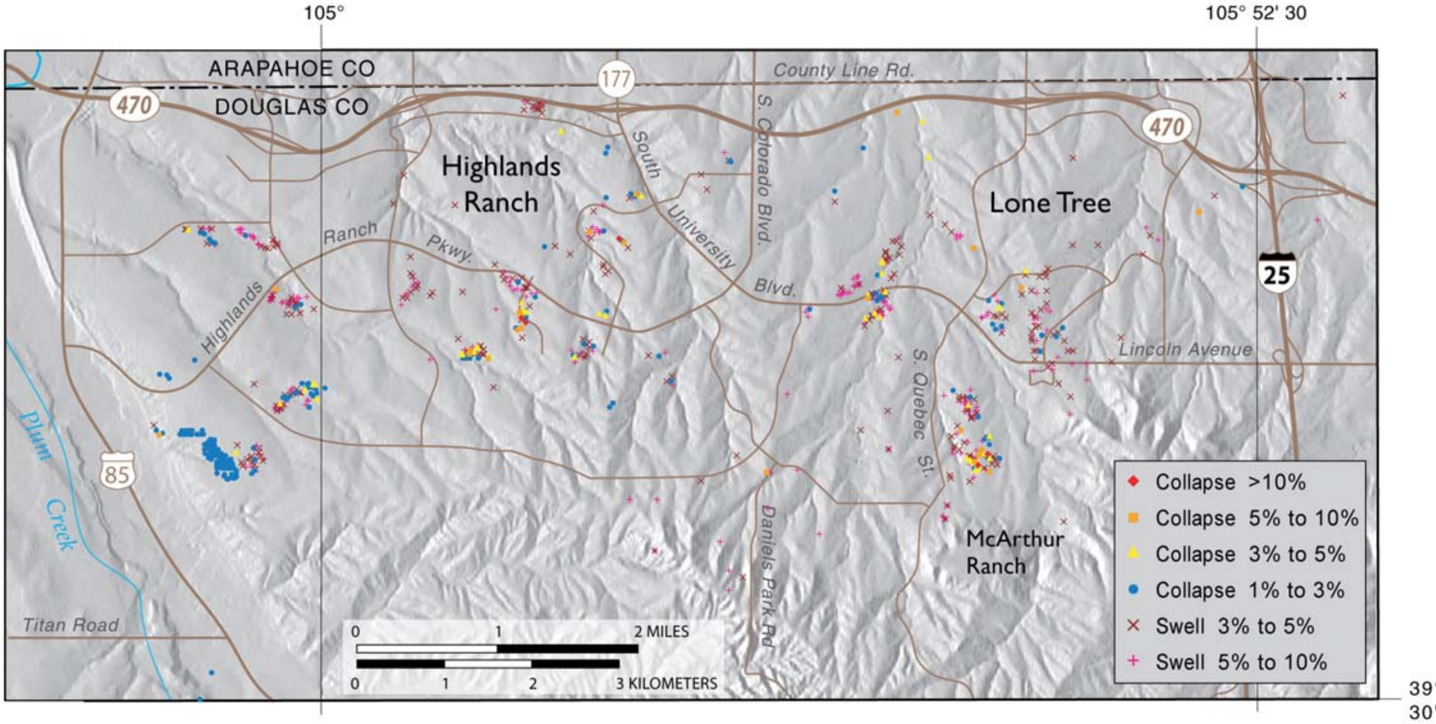


Figure 7-5. Locations of swell/consolidation test results in northwest Douglas County. Note the proximity of highly collapsible and swelling soils. Data are from swell/consolidation test results when the soil is loaded to 1,000 psf and then wetted.

encountered at Truckton in eastern El Paso County during investigations for a maintenance building. A sample there showed 5 percent collapse at 1,000 psf. In both cases, the foundation recommendation was spread footings on 2 ft of recompacted soil. These structures are at risk because the cumulative impact of hydrocompaction of 25 ft of eolian soil with 3 to 5 percent collapse potential, perhaps as the result of a drainage problem, would be significant.

Piedmont Alluvial Deposits

Slack-water or low-energy alluvial sediments on flood plains may also be collapse prone in the Colorado Piedmont area. These sediments can be found in drainage-channel bottoms and in late Pleistocene and Holocene terraces. Collapsible soils derived from alluvial sediments have been mapped near Golden and in the Parker area (Simpson, 1973; Maberry, 1972) and have been documented in the drainages and terraces of northern Douglas County (Berry and others, 2002). Piping erosion and collapse have also been documented in alluvium of reworked loess along drainage gullies in the grasslands northeast of Greeley (Brown, 1962).

The Pleistocene terrace and pediment deposits, such as the Slocum Alluvium and Verdos Alluvium, are commonly present as eroded remnants along the Front Range; these deposits contain a fine-grained fraction that could be hydrocompactive. Collapsible behavior is also seen along tributary drainages of the South Platte and Arkansas Rivers, where the late Holocene Piney Creek Alluvium is widespread as a younger terrace and as infill in smaller drainages. Likewise, erosion of Cretaceous formations such as the Pierre Shale, Laramie Formation, and Denver Formation resulted in deposition of clay-rich alluvium in many areas, filling tributary drainage valleys with dry, fine-grained, low-density, collapse-prone deposits.

In the Pueblo and Cañon City Embayment areas, along tributaries of the Arkansas River, substantial thicknesses of alluvial sediments are found. Boring logs from consultant reports on the western side of Pueblo reported as much as 60 ft of undifferentiated alluvium composed of interbedded clay, silty sand, and sand that show collapse behavior. Erosion in these alluvial deposits near Pueblo and Cañon City can be severe, and arroyos are common.

The *Soil Survey of Pueblo Area* described Limon soil, which is formed from Piney Creek Alluvium, as characterized by soil piping and deep gullying (Larsen and others, 1979). The soil is described as "difficult to till because it is cloddy when dry and sticky and plastic when wet." Like some of the clayey eolian sediments of the Colorado Piedmont, the Piney Creek Alluvium may be both expansive and collapsible. The soils with swell properties tend to have higher dry densities (greater than 100 pcf), higher plasticity, and less silt than those with collapse behavior. Conspicuous rolling in the roadway can be seen and felt when driving east on Highway 50 from Pueblo, which could be caused by soil expansion, collapse, or both in the underlying Piney Creek Alluvium. Another case history involves the Kaiser Aerospace facility in the Pueblo Airport Business Park, which was damaged considerably where collapsible soils beneath the entryway were fairly closely constrained by expansive soils and stable soils. Soil collapse at this site was initiated by a leak in the building air conditioner that caused water to pond beneath a corner of the building. The 65-ft thickness of sediment at Kaiser Aerospace made installation of deep foundations costly, so the facility was built on shallow foundations. The remedial mitigation consisted of installing pin piers around the building and jacking up the building and clipping it to the piers, a mitigation method that will be further discussed in Chapter 9. No damage has yet been reported from other large commercial buildings in the business park with similar foundations.

Upstream on the Arkansas River, in the Cañon City Embayment, the thickness of younger tributary alluvium ranges from a few feet to about 35 ft. The alluvium was mapped as Piney Creek by Scott (1977) and as Limon soil by the NRCS (Larsen and others, 1979). The manifestations of collapse in the soil can be seen at a number of locations where case histories exist. At Harrison Elementary School in Cañon City (fig. 3-7A), the brick building was constructed in the 1920s and shows extensive cracking, even through the foundation wall. Just southeast of Cañon City, severe damage from both swelling and collapsible soils occurred at the Arrowhead Correctional Facility. The facility, built in 1991 on shallow foundations, is located on soils mapped as Piney Creek Alluvium, which might include slope-wash deposits derived from the Pierre Shale and the Verdos Alluvium. Cracks began to appear while the contractor was still on site. By 1999, four buildings showed significant damage. The damage was so severe that the administrative building had to be demolished. Total repair costs were almost $3 million to underpin three buildings and demolish and replace the administrative building with a precast structure on deep foundations. The forensic investigation indicated that collapsible soils were deep, located below swelling soils and closer to bedrock.

Piedmont Colluvial Slope-Wash Deposits

The Dawson Formation and Castle Rock Conglomerate of Douglas, Elbert, and El Paso Counties and the pre-Tertiary sedimentary formations in the foothill hogback regions erode to form sheets of colluvial soil on the slopes of hills, bluffs, ridges, and small buttes. Complex interbedding, intermixing, and reworking of eolian deposits may contribute to the unconsolidated deposits. Some colluvial soils of the Front Range area of the Colorado Piedmont also have swelling clay. Both collapsible and swelling colluvial soils have been reported in proximity to eolian soils in northern Douglas County (Berry and others, 2002).

An area of construction problems that has been widely documented is the Austin Bluffs region of Colorado Springs, where collapse of colluvial soils reworked from eolian deposits and erosion of the Dawson Formation occurred at the Montbello Drive Post Office, Grant Elementary School, and the Commercial Center at Academy and Flintridge Boulevards. Many problems in this area were thought to result from the rise in ground-water levels into previously unsaturated, low-density soils, related to increased urbanization (and irrigation) of eastern Colorado Springs (George Morris, personal commun., 1997). Low-density and potentially hydrocompactive soils are present in the colluvial wedges within the strike valleys of the Front Range hogbacks near Loveland and in the Colorado Springs area (John W. Himmelreich, Jr., personal commun., 2000).

ROCKY MOUNTAINS PROVINCE

The central Rocky Mountains of Colorado are composed of (1) Blocks of Paleozoic and Precambrian sedimentary, igneous, and metamorphic rocks that started thrusting upwards about 65 million years ago, and (2) the Tertiary San Juan volcanic rocks of southwest Colorado. Thus the province consists of hard, resistant, rock formations at high elevations except for isolated structural basins. The mountains are snowpacked for much of the year, and the annual precipitation is greater than 20 in. The saturated soils found in this climate zone are not prone to collapse. Certain structural basins adjacent to the mountain ranges, such as the San Luis Valley and South Park, have both a dry climate and also Holocene sediments, but the type of source rock limits the formation of collapsible soils. The predominantly rural areas of these basins have yielded little data on engineering properties of the soils. This situation contrasts with that of the intermontane valleys discussed next.

Mature Intermontane Valleys of West-Central Colorado

The intermontane river valleys of the Colorado River Basin, such as the lower-elevation, semiarid corridors of the Colorado, Eagle, and Roaring Fork Rivers, contain significant areas that are prone to the formation of collapsible soils (Robinson and Miller, 1977; Morris and Weave, 1978; Mock and Pawlak, 1983; Pawlak, 1998; White, 1998, 2001, 2002).

These intermontane river valleys drain the lower reaches of the central Rockies, from west of the

Continental Divide to the Grand Hogback (the westernmost structural feature of the Rocky Mountains Province) (**figure 7-6**).

The intermontane valleys are quite wide, and the valley floors are terraced with Pleistocene glaciofluvial gravelly outwash. The bedrock is composed of the Maroon Formation, Eagle Valley Evaporite, and Eagle Valley Formation, which are nonresistant and form low hills that have been beveled by erosion and pediment formation during the different glacial ages. The sediments derived from these rocks have been deposited as hillside colluvium, colluvial sheetwash, alluvial fans, and valley fill and form some of the most problematic, collapse-susceptible soils in Colorado (**figure 7-7**). Early alignment studies of Interstate 70 in the Eagle River valley noted the hydrocompaction potential on alluvial fans of the area (Ken R. White Company, unpublished report for CDOT, 1966). The final soils and geology report for the proposed Interstate 70 alignment mentioned that the old U.S. Highway 6 embankment settled about 1 ft following hydrocompaction of 30 ft of alluvial-fan soils, 2 mi west of Gypsum.

Many case histories of collapsible-soil occurrences generated for this publication are in the Eagle and Roaring Fork River corridors, where development is widespread and growth continues (**figure 7-8**). The figure 5-5B terrain model that includes the alluvial fan shown in figure 7-7 represents the geomorphology of the surficial soil deposits in these intermontane valleys. In the intermontane valleys, major towns that have experienced problems with hydrocompactive soils that can be related to Eagle Valley Evaporite include Glenwood Springs in Garfield County and Basalt, Gypsum, Eagle, Edwards, and Avon in Eagle County (town locations are shown in fig. 7-6).

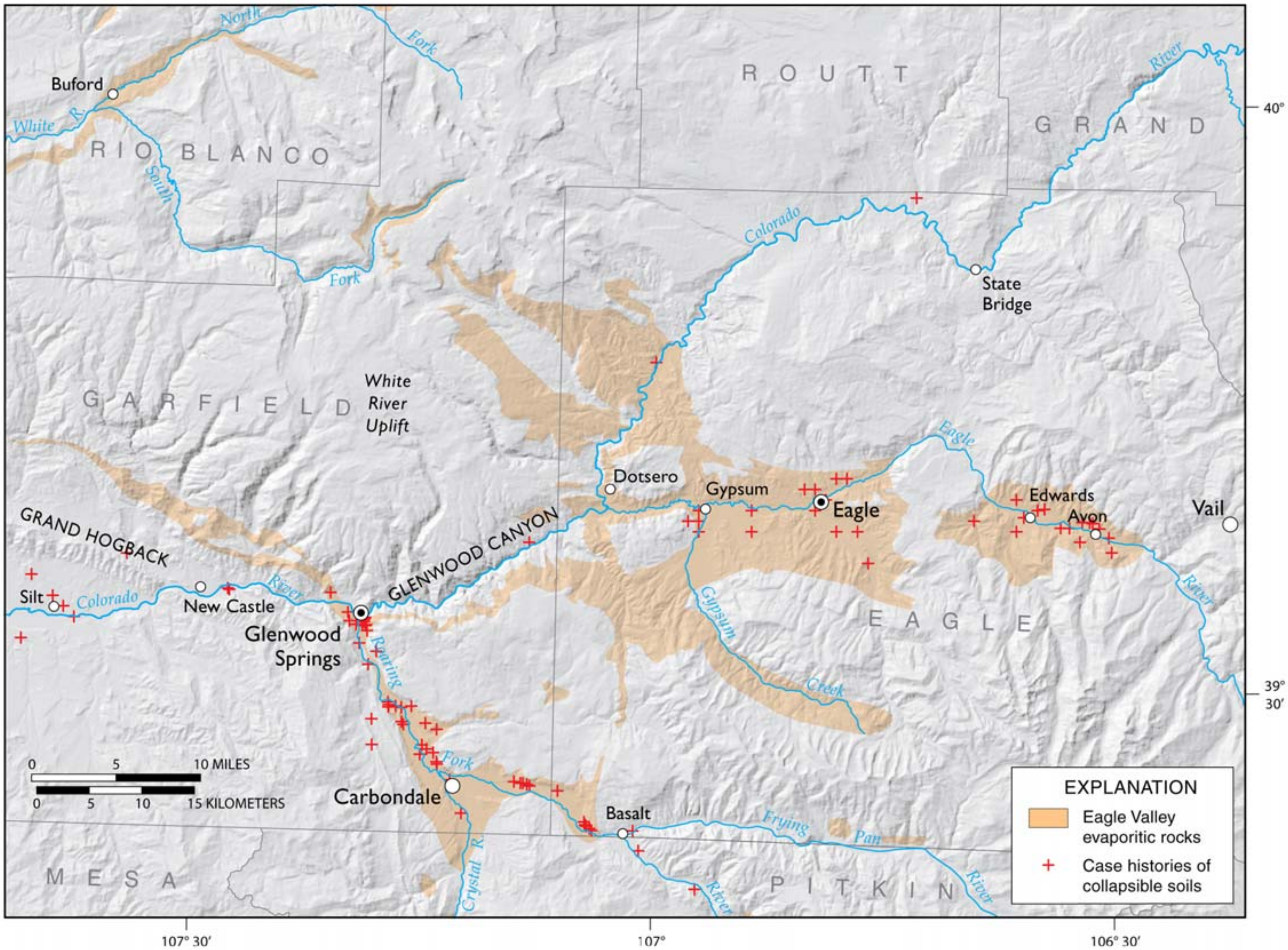


Figure 7-6. The extent (tan-shaded areas) of the Eagle Valley Evaporite and Eagle Valley Formation, either at the surface or under thin surficial deposits. Points are locations of collapsible-soil case histories.

Figure 7-7. Aerial view southward of an alluvial fan (shown by dashed line) on the floor of the Roaring Fork River valley between Glenwood Springs and Carbondale. Carbondale is up valley to the left. These collapse-susceptible fan deposits may be as much as 100 ft thick. Smaller coalesced colluvial wedges of sediments that mantle the base of the valley walls above and left of the fan are also highly susceptible. The collapse-susceptible areas on this fan are currently under residential development, and damage from ground settlements has occurred.

Figure 7-8. Aerial view of recent development in the town of Gypsum. Note the poorly vegetated hills of Eagle Valley Evaporite bedrock and the heavily irrigated residential lots. The soils mantling the hillsides that underlie this development are susceptible to collapse.

Upper Arkansas River Valley

The Arkansas River rift valley around Salida and the lower Arkansas River Canyon fall within the semiarid precipitation limit (pl. 1). Towns potentially underlain by collapse-susceptible soils include Salida, Wellsville, Howard, and Coaldale. Collapse-susceptible soils can occur in the alluvial-fan sediments in these areas, as well as on colluvial slopes where the soils are derived from sedimentary rocks, some of which are evaporitic. There are occurrences of collapsible soils and settlement damage along the margins of Salida above the modern river valley bottom, where fine-grained alluvial-fan and colluvial sediments, derived from the Tertiary Dry Union Formation, have accumulated (T.C. Wait, personal commun., 2005).

COLORADO PLATEAU AND WYOMING BASIN PROVINCES

The Colorado Plateau Province is characterized by predominantly flat-lying or slightly dipping sedimentary rocks that have been eroded to create the high plateaus, mesas, and cuestas typical of the area. Many of the formations exposed in the plateau area are Cenozoic and Cretaceous, poorly indurated, clay- and silt-rich rocks. Only isolated highland areas such as Douglas Pass, the Roan Plateau, the Uncompahgre Plateau, Grand Mesa, and Battlement Mesa receive more than 18 in. of annual precipitation. The remainder of the Colorado Plateau Province—the Four Corners area, the Grand Valley, and areas west of Rangely—is semiarid or arid in climate and receives as little as 8 in. in average annual precipitation.

In the Colorado Plateau Province, most geologically recent clay- and silt-rich surficial sediments associated with the depositional systems discussed in Chapter 5 are susceptible to collapse by classic mechanical hydrocompaction, by dispersion and (or) piping erosion, or (if very clayey) by hydrocompression

Eolian deposits in the Colorado Plateau Province are scattered erosional remnants of middle to late Pleistocene loess blankets that mantle mesas and other flat-lying areas. Although the loess deposits were probably widespread at the end of the Pleistocene, most have subsequently been eroded. Soils formed from loess deposits of western Colorado generally have a reddish hue and a developed soil horizon that indicate ages in the middle to late Pleistocene (Price and others, 1988; Shroba and Birkeland, 1983; Shroba, 1984). More recent (Holocene) windblown deposits occur in some areas. Thin deposits have been reported by Luehring (1988) in his work in southwest Colorado south of Cortez, where he described thin loess sheets interlayered with fine-grained alluvial-fan deposits (mud). This common depositional relationship on alluvial fans in arid environments was also reported by Beckwith and Hansen (1989).

The thickest blanket of loess in western Colorado is near Dove Creek in Dolores County (Price and others, 1988). Other significant loess blanket deposits are widespread in the Glade Park area on the Uncompahgre Plateau above the Colorado National Monument (Spears and Kleven, 1978; Scott and others, 2001).

Where the major river systems of western Colorado exit the Rocky Mountains Province and enter the Colorado Plateau Province, the river gradients flatten and the valleys widen. The three major river systems of western and northwestern Colorado are the Colorado, White, and Yampa. The semiarid climate of this region results in an annual precipitation of 10 to 15 in. or even less in areas west of Rangely. The geologic units are the Upper Cretaceous Mancos Shale and Mesa Verde Group and Tertiary strata, such as the Browns Park Formation, Wasatch Formation, Green River Formation, and Uinta Formation, which are all poorly indurated sedimentary rocks with high clay and silt contents (fig. 5-4). The climate and rock types facilitate the formation of collapse-prone soils

The floors of the main river valleys are terraced with outwash and fluvial gravel from the Pleistocene glaciation of the Rocky Mountains. These Pleistocene sediments can be mantled with variable alluvial-fan and colluvial sediments eroded from the valley sides and from ephemeral streams. The tributaries are second-order streams that do not originate in glaciated terrain. The tributary basins are generally located within Tertiary or Cretaceous sedimentary rock terrane that erodes to fine-grained sediments. Piping erosion and arroyo formation are common in the resulting Holocene alluvial sediments. The dissected remnants of older middle to late Pleistocene erosional surfaces commonly form gently dipping mesas along the flanks of the river valleys. These mesas are typically mantled with middle to late Pleistocene loess (Soule and Stover, 1985; Scott and Shroba, 1997; Shroba and Scott, 1997)

Grand Valley and Lower Uncompahgre Valley

The Grand Valley and the lower Gunnison River and Uncompahgre River valleys form a wide broad belt of subdued terrain that extends from the Utah border through the Grand Junction area, toward Delta and Montrose. This geomorphology is the erosional expression of the weathering and lowering of thousands of feet of the Mancos Shale, as seen in a shaded relief map (**figure 7-9**). The broad valley is, on average, 13 mi wide and is a major landmark of western Colorado.

Typically, the surficial geology of the Grand Valley and lower Uncompahgre Valley is composed of the following: (1) highly dissected early to middle Pleistocene gravel-capped mesa remnants; (2) middle-

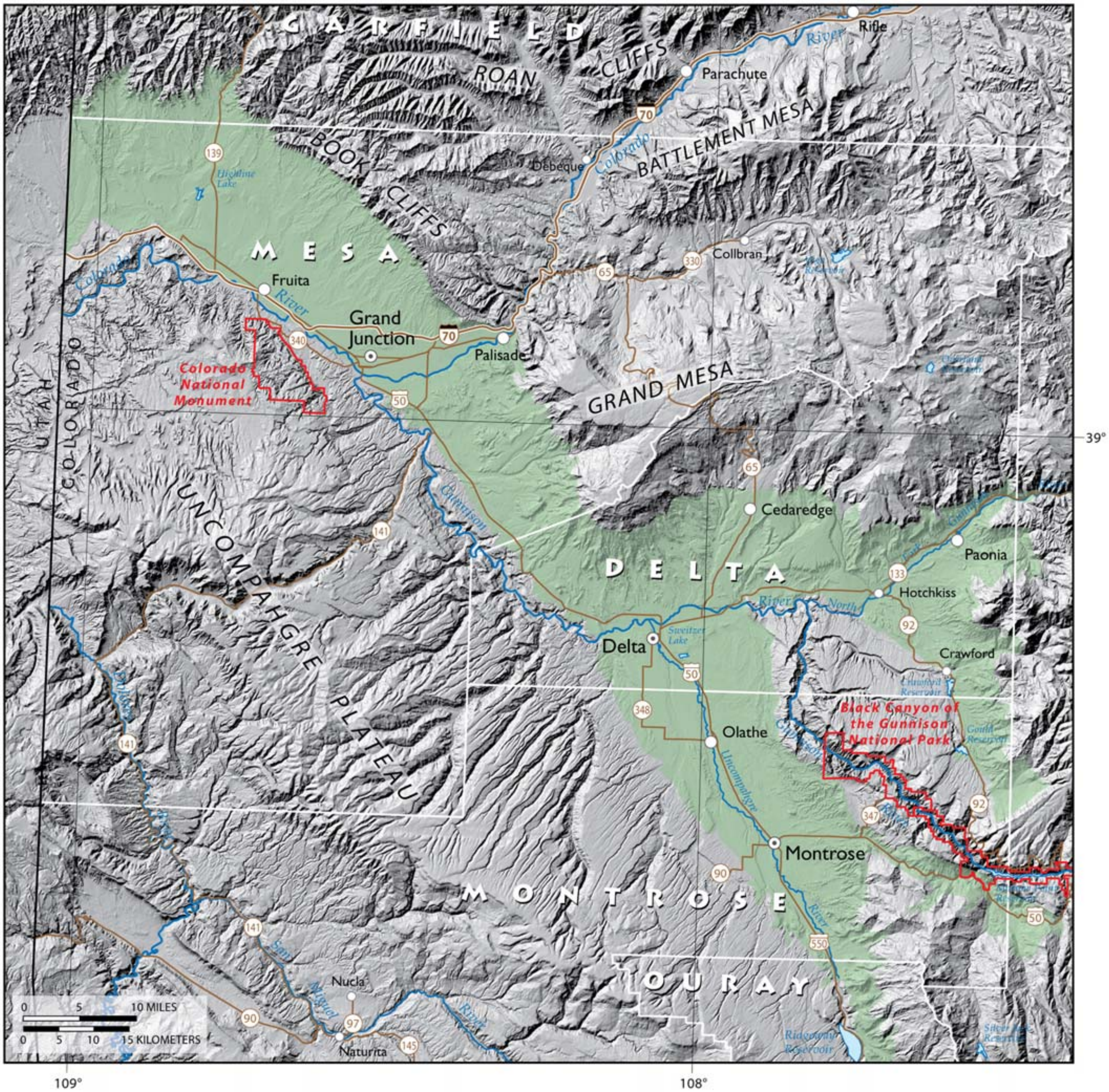


Figure 7-9. The green areas show the extent of the Mancos Shale in west-central Colorado, modified from Tweto (1979).

and lower-elevation mesas capped with late Pleistocene graveled terraces or subtly tilted pediment deposits; (3) the dendritically incised “adobe” hills of exposed, weathered, “Blond,” Mancos claystone (**figures 7-10** and 5-3A); and (4) Holocene alluvium of silty clay, eroded from the Mancos Shale, that has formed discontinuous but widespread mud-flow fans, playas and mud flats, overbank deposits, and low-lying pediments.

The highest susceptibility to collapse is associated with low-level pediments and alluvium. The higher and older pediments and terraces are capped with gravels derived from erosion of the hard-rock terrane of Grand Mesa, the San Juan Mountains, and the Black Canyon uplift area, whereas the lower-level alluvium deposits are derived from more recent mud flows and overbank mud sediments eroded from Mancos Shale and (or) the Mesa Verde Group (similar soils are found in Montezuma Valley and the Rangely area). These are fine-grained alluvial sandy-clay and silt soils that can have high sodium and sulfate concentrations. In addition to potential collapse, hydrocompression, and dispersion, soil derived from Mancos Shale can be expansive and

Figure 7-10. Pediment surfaces underlain by Mancos Shale on the western flank of Grand Mesa. The pediments are sloped toward the Gunnison River near Whitewater. The San Juan Mountains are in the background. In the bottom left corner of the photograph, note the weathered rind of "Blond" Mancos immediately below the pediment surface.

problematic for introducing salts into the Colorado River system (Butler, 2001). It is an increasing concern that irrigation water runoff from these soils is having adverse affects on water quality.

Older, heavy structures in Montrose, Olathe, Delta, and Grand Junction commonly have experienced various forms of distress caused by collapse and compression of the soils upon wetting. Several were condemned and demolished.

Many of these clay-rich, gypsiferous soils eroded from the Mancos Shale contain swelling clay and show both expansive and collapse characteristics. Geotechnical investigations have revealed surprising changes in the structure of these soils related to previous irrigation and biotic activity, which can increase the collapse potential. In many areas of the Grand Valley and lower Uncompahgre Valley covered with Holocene alluvium and late Pleistocene pediments, ground that was once irrigated and then left idle has increased collapse potential when placed under structural loads. Secondary macroscopic porosity that formed coincidentally with irrigated agricultural use is responsible for the additional collapse under structural loads. The secondary voids created during long-term wetting periods are related to worm burrows, root penetration, microdispersion, gypsum dissolution, and micropiping enlargements that were discussed in Chapter 4. Geotechnical investigations describe the pedogenic voids as visible vesicular-like pores, referred to as "sponge-type voids." The dry, vesicular soil sample in figure 3-3B is an example recovered on Garnet Mesa, near Delta, from previously irrigated fields; the soils there collapsed more than 10 percent when wetted at 1,000 psf during consolidator testing (Laurie Hauptmann, personal commun., 2002).

The arroyos in the clay- and silt-rich, dispersive alluviums of the area, such as the Loutsenhizer and Montrose arroyos, have the typical pseudokarst morphology in which many fissures and ground openings can extend more than 200 ft from the arroyo channel. Ground openings have affected roadways in the area (figure 5-11), and local stories allege that agricultural equipment has spontaneously fallen into previously unknown voids when the soil bridges above the voids collapsed. At the Montrose County airport, close attention was paid to engineering the improved runway that crosses Cedar Creek arroyo. Fissures and soil sinkholes had been noted in the airport area, and no risk was acceptable for the potential of spontaneous openings in the runway. Subsoils along the runway alignment were overexcavated and replaced at 10:1 (horizontal to vertical) grades from the bottom of Cedar Creek arroyo to ensure that no subsurface voids remained along the runway alignment (T. Kearney, personal commun., 1998).

The Mancos Shale is not the only source rock for collapsible soils in the Grand Valley. At the base of the Colorado National Monument, red clayey silts and sands, known as the Redlands alluvium, were deposited in successive debris and mud flows that fanned from older red formations exposed in the monument (the soil sample shown in fig. 3-3A is a sample of Redlands alluvium). These soils have a different provenance and higher silt and sand content than soils derived from the Mancos, but are also very prone to mechanical collapse and piping.

Paradox Salt Anticline Region of Southwest Colorado

The Paradox salt anticline region lies in southwestern Colorado in Mesa, Montrose, and San Miguel Counties (lower left corners of figs. 5-16 and 7-9). A series of northwest-trending valleys and higher basins include Big Gypsum Valley, Paradox Valley, and Sinbad Valley. These valleys are floored with evaporite rocks of the Paradox Member of the Hermosa Formation. The geologic structure is complex and is related to diapiric upward movements of evaporite deposits, which caused folding and faulting of the overlying formations (Cater, 1970). The result is structurally breached, anticlinal valley floors, with parts of the valley walls composed of evaporite bedrock. This part of the State is arid; its annual precipitation is only about 12 in. Typical evaporite karst landforms (e.g., sinkholes) are present (Cater, 1970), and the colluvial and alluvial soils derived from the Paradox Member are also highly susceptible to collapse.

In this rural arid area of southwestern Colorado, it is unlikely that broad residential use of the land will occur at any time in the near future. However, the geomorphic conditions are present for highly collapsible soils. State Highway 141, which is one of the few paved roads through this area, shows distress from differential hydrocompactive settlement where it crosses collapse-prone alluvium and alluvial fans (**figure 7-11**).

Colorado River Between New Castle and Debeque

Past the water gap through the Grand Hogback, the Colorado River enters the Piceance Basin, a part of the Colorado Plateau Province in which Cenozoic sedimentary formations are relatively flat lying. The north valley side is bounded by the Roan Cliffs, which are capped by the Green River Formation, with dissected foothills of Wasatch Formation near the base. Erosion of middle Pleistocene pediments and broad alluvial fans that extend from Battlement Mesa has created high mesas on the south valley side. Rainfall averages about 12 in. per year, and the vegetation consists of spotty sagebrush and grasses with isolated juniper trees. Ground conditions include badlands and abundant bare soil. Collapsible soils are commonly found in the following deposits: (1) broad alluvial fans from small drainage basins in the highly erodible formations (**figure 7-12**), (2) thin erosional remnants of loess on the higher mesas along the Colorado River (Soule and Stover, 1985; Shroba and Scott, 1997; Harman and Murray, 1985), (3) hillside colluvium derived from the Wasatch Formation, and (4) most of the lower reaches of tributaries of the Colorado River that contain fine-grained alluvium on the valley floors. Examples include the Government Creek arroyos along Highway 13 from Rifle to Rio Blanco Hill; the Mamm, Dry Hollow, and Divide Creek basins south of the Colorado River near Silt; and the Roan and Parachute Creek valleys.

Major towns of Garfield County lie along this stretch of the Colorado River. New Castle, Silt, Rifle, and Parachute–Battlement Mesa all have reported incidents of collapsible soil, where structures have been damaged and required mitigation and repair. The Garfield County School District is currently replacing schools in Silt because foundation settlements have damaged existing facilities. Some of the alluvial-fan soils in the town of Silt are more than 80 ft thick. The upper 15 to 20 ft of the sediments are dry and highly hydrocompactive, but they are moist, very soft, and highly compressive at greater depth. All new schools will be built on deep foundations that will bear on either dense river gravels or bedrock below these problematic soils. The collapsible nature of these young alluvial-fan sediments was also verified in geotechnical investigations during the planning, design, and construction of Interstate 70 in western Colorado

Figure 7-11. State Highway 141 shows settlement distress related to hydrocompaction where it enters Big Gypsum Valley and passes over collapse-susceptible alluvial fans and Big Gypsum Creek alluvium (low-relief, vegetated areas on the left and right of the road). Evaporite (gypsum) bedrock is exposed on the hill to the right.

Figure 7-12. A large (1 mi wide) alluvial fan (shown by dashed line) debouching from Smith Gulch 5.5 mi west of Parachute has diverted the Colorado River. The lower, red-banded hills are the Tertiary Wasatch Formation. The upper, light-gray cliffs of the Roan Plateau are the Green River Formation. Another alluvial fan about 7 mi upstream along the Colorado River is shown in figure 5-6A.

that was discussed in Chapter 6. Swell/consolidation soil testing by CDOT showed significant collapse; one soil sample collapsed an astonishing 25 percent when wetted at a load equivalent to the overburden pressure (2,000 psf) that would be exerted by the planned fill thickness for construction of the highway embankment (Shelton and others, 1977).

Northwest Colorado: Moffat and Rio Blanco Counties

The plateau region of northwestern Colorado includes the low-lying areas along the White and Yampa Rivers and the major tributaries of Piceance Creek, Douglas Creek, Stinking Water Creek, and Little Snake River. Vermillion Creek is the main tributary of the Green River in Colorado, north of Dinosaur National Monument. The climate is primarily semiarid, with annual precipitation of about 15 in. at Meeker and Craig near the western edge of the White River uplift, decreasing to 10 to 12 in. at Rangely and Dinosaur near the Utah border. Precipitation increases in the higher elevations toward Douglas Pass and the Roan Plateau to the south and in the Danforth Hills and White River uplift to the east.

The geologic structure is relatively undisturbed outside of the Dinosaur National Monument area except for some structural warping of the rock formations that created the oil reservoir of the Rangely field. The Upper Cretaceous Mancos Shale and Mesa Verde Group and the Tertiary formations are the source materials for sediments that may be collapsible.

The major landforms where collapsible soils form are alluvial fans, colluvial sheetwash wedges that mantle the edges of the river valleys and adjacent hillslopes, and the valley-bottom terraces where recent fine-grained alluvium has aggraded along the tributary streams of the area. Pleistocene gravel terraces are present along the main trunk of the White River, but there are insufficient highlands to generate high pediment surfaces. Overlying the river gravels are alluvial fans that accumulate at the mouths of ephemeral streams flowing from the adjacent hills (**figure 7-13**). Soil fissuring, piping erosion, and arroyo formation are also common in the terraces of the tributary streams, which are composed of fine-grained sediment from the local bedrock sources.

The towns of Meeker, Rangely, and Dinosaur have experienced severe problems due to collapsible soils.

Figure 7-13. This meander of the White River has cut into recent alluvial-fan sediments that were deposited onto the valley floor from an ephemeral tributary stream. This dry soil, derived from the Wasatch Formation, should be considered susceptible to soil collapse, if wetted.

Significant parts of the town of Meeker lie on coalesced alluvial fans and sheetwash deposits originating in China Ridge, which is the northern extent of the Grand Hogback where Mesa Verde Group rocks are exposed. The newer, western third of the town, which was built out in the late 1970s and early 1980s during the energy boom, lies on a large alluvial fan that slopes up to the mouth of Anderson Gulch (**figure 7-14**). The thickness of the alluvial-fan soils exceeds 70 ft. The collapse potential of the alluvial-fan soils has created severe problems for Meeker. Several homes and apartment buildings have required remedial repair work or have been condemned (**figure 7-15**) and removed because of damage caused by hydrocompactive soils. In addition to localized house-specific collapse, large-scale subsidence, more than 200 ft across, has also occurred in town, related to water-main breaks and moisture migration in more permeable, gravelly lenses in the alluvial-fan deposits (**figure 7-16**). Nearby, alluvial-valley fill of Sheep and Strawberry Creeks is also gullied and prone to piping erosion and collapse.

The other main town in Rio Blanco County, Rangely, also lies on a broad alluvial fan on the south valley side of the White River. Thick accumulations of fine-grained sediments eroded from the Mancos Shale and Mesa Verde Group rocks overlie buried White River terrace gravels in the area. The soils, because of the influence of the expansive clays in the Mancos Shale, tend to be clayey with the typical properties of clay-rich collapsible soils: slightly expansive under no loads, or very light loads, but becoming increasingly hydrocompressible once wetted under increasing load. Along the town's main street, these soils are about 30 to 40 ft thick. The town of Rangely, like Meeker, has had a history of problems related to hydrocompactive and compressible soils. Most of the heavily loaded structures, such as schools, the town municipal building, and multistory brick apartment complexes, have had damage that required remedial repair. For some structures, several inches of total settlement occurred, resulting in so much damage that demolition was considered. Nearly all of these structures were built in the 1970s and early 1980s, and design and drainage errors occurred in their construction. Heavily loaded structures with spread footings or with friction piers that did not extend through the collapsible-soil column generally incurred the most significant distress. Compaction grouting—in which a subsurface grout column was injected through the collapsible-soil mantle to support the original foundation walls of the structures—was used successfully to mitigate damage to some of the more prominent buildings in town (Sam Bandimere, personal commun., 1998). More discussion on the compaction-grouting mitigation technique is included in Chapter 9.

Most of the Rangely damage was settlement that resulted from poor drainage and general deep wetting of the dry collapse-prone subsoils. In addition, there

Figure 7-15. This home in Meeker was condemned and subsequently demolished because of settlement of hydrocompactive soils. Note the gap between the subsided foundation and the home's exterior wall.

Figure 7-14. The town of Meeker, with view to the northwest. The mouth of Anderson Gulch is shown by the white arrow.

Figure 7-16. Severe settlement seen in more wide-scale subsidence in Meeker. Settlement of several feet has occurred from deep wetting of thick sections of collapse-prone alluvial-fan soils. *A*, Settlement and tilt of an abandoned sidewalk on the margin of a large subsidence area. Home to the right of the side-walk (not shown in photograph) was demolished because of the extent of the damage due to differential settlement. *B*, Across the road on the opposite side of the subsidence area, the offset and tilt of the concrete walkway shows the extent (greater than 4 ft) of settlement of the roadway compared to the home. Note large stone used for additional step.

are unsubstantiated reports that the collapse and settlement of the local soils was exacerbated by seismic shaking caused by low-intensity earthquakes induced by the tertiary recovery injection program at the Rangely oil field. This theory involves a two-step process in which soils were first wetted to the degree of saturation at which grain-to-grain contacts of the soil fabric weaken but are still strong enough to resist grain-to-grain shearing and densification of the soil. Then, the peak particle velocity from the small earthquakes possibly caused the already-strained soil-grain contacts to fail, which produced collapse and relatively rapid settlement shortly after the shocks were felt.

Conspicuous damage is also present along the roadways in the Rangely area. State Highway 64 from Meeker to Rangely follows the White River valley and crosses several alluvial fans and overbank alluvium deposits of intermittent and ephemeral tributary streams. In several locations, downwarping and distress of pavement can be observed (**figure 7-17**). The piping erosion and gullying of the thick valley fills of Douglas Creek have affected State Highway 139 south of Rangely. More prominent distress to roadways can be seen along County Road 1 from the Rangely oil field to Blue Mountain, locally called the Blue Mountain Road. This road follows Stinking Water Creek, where thick,

Figure 7-17. Settlement distress on State Highway 64, between Meeker and Rangely, where it crosses over coalesced, collapse-prone alluvial-fan and hillside colluvial soils on the floor of the White River valley.

Holocene, fine-grained overbank alluvium and sheet-wash colluvium overlying Mancos Shale are currently being incised by arroyos as the creek down cuts (see photograph in fig. 5-10C). As evidenced by the creek name, there is high salt content to the soils, which are both hydrocompactive and dispersive. Ground cracks, fissures, and subterranean voids form where the soils have washed away. Dips, sags, undulations, and washouts of embankments at gully heads near the roadway have required constant maintenance by the Rio Blanco County Road and Bridge Department.

The town of Dinosaur and Dinosaur National Monument are located in Moffat County, in the same physiographic province as Rangely, but immediately adjacent to the structural uplift of the Uinta Mountains. Dinosaur is located on surficial deposits at the mouth of ephemeral streams emanating from small basins in the adjacent hogbacks of Cretaceous strata along the southern limb of the uplift, beginning with the Frontier Member of the Mancos Shale. The clayey to sandy-silt soils derived from the hogbacks are prone to hydrocompaction, and several structures in town, including the school and town hall, show evidence of settlement distress.

The Dinosaur National Monument headquarters is located about 2 mi east of Dinosaur on U.S. Highway 40. The headquarters and maintenance buildings lie on alluvial-fan and sheetwash colluvium where Dripping Rock Creek exits the Frontier sandstone hogback. Reportedly, windblown sand and loess also are interlayered in the soils. Both the headquarters and the maintenance buildings have experienced significant distress due to settlement of hydrocompactive soils. The maintenance building, founded on shallow spread footings, lies on a mantle of hydrocompactive surficial soils that varies from 7 ft to more than 40 ft in depth. This difference in thickness of the collapsible-soil column resulted in significant differential settlement. The entire building "unhinged" at the center as one side dropped several additional inches compared to the other (**figure 7-18**).

Montezuma Valley and Sleeping Ute Mountain Indian Reservation

Montezuma Valley is located just south of Cortez in Montezuma County. The broad valley includes low, subdued, eroded areas (underlain by the Mancos Shale) below the northwestern rim of Mesa Verde, a geomorphology similar to that of the Grand Valley near Grand Junction. The valley sides and floor are mantled by variable thicknesses of recent alluvial sediments. These have washed in from the adjacent higher terrain, which is composed of shale and sandstone of the Mesa Verde rim and pediments and highlands of Sleeping Ute Mountain. The town of Cortez is sited on Dakota Sandstone bedrock that is near the surface and variably mantled by loess. This loess blanket thickens to the northwest along U. S. Highway 491 toward Dove Creek.

Various soils of Montezuma Valley and the Navajo Wash area have presented many problems from soil collapse over the years. Collapsible soils are prevalent in four types of deposits:

1. Fine-grained mudflows in alluvial-fan deposits and slope-wash sediments are derived from exposed Mesa Verde Group and underlying Mancos Shale. Soils in these settings are the most collapse prone in the area.

2. Alluvial-fan sediments with a high gravel and cobble content are found along the western flank of Montezuma Valley near the town of Towaoc. These deposits are derived, in part, from erosion of older gravel-capped pediment remnants sloping from Sleeping Ute Mountain, many of which are also mantled with thin loess sediments).

3. Recent fine-grained alluvium forms some of the floors of drainageways incised into the bedrock. The drainage bottoms of Navajo, Aztec, Cowboy, and Mariano Washes, and McElmo Creek Canyon, which is north of Sleeping Ute Mountain, are partially filled with fine-grained alluvial soils that have shown the tendency to settle, disperse, and pipe when wetted. Most of the bottomlands along these washes show pseudokarst morphology typical of soil dispersion and piping. Figure 3-5 contains illustrations of a piping-erosion example on U.S. Highway 160 at Aztec Wash, in the extreme southwest corner of Colorado. These soils contain smectite clay so can also be expansive, similar to soils in the Grand Valley and Rangely areas.

4. Thin loess soils on older terrace and pediment mesas south of Sleeping Ute Mountain in the Four Corners region have been shown to contain heavy gypsum concentrations (Doug Ramsey, personal commun., 2001).

The problematic nature of these soils has come under increased scrutiny since the late 1980s. The Ute Mountain Tribal Authority has had significant problems with many structures on tribal lands, in both the fine-grained soils of alluvial fans and the more clast-rich, matrix-supported sediments along the mountain flank. In 1989 the Tribal Housing Authority implemented a stabilization project for 60 homes that were damaged as a result of collapsible and swelling soils. A tribal convenience store and self-service laundry founded on shallow spread footings at the intersection of Towaoc Road (Indian Route 202) and U.S. Highway 160 was damaged so badly that it was condemned and demolished in 1997. Settlement of 17 to 18 in. in homes constructed on sandy-silt and clay soils drove the Tribal Housing Authority to ban all residential construction in the lower valley area near Highway 160 (Ben Cordova, personal commun., 1997). The USBR also is cognizant of the collapse hazards because of their irrigation canals in Montezuma Valley. The documented settlement of 3 to 4 ft at an irrigated, 100-acre agricultural plot on Ute tribal lands prompted the study for the Towaoc Canal Reach 2 and 3 alignments by the USBR (Luehring, 1988). Road and bridge crews have long recognized the maintenance issues of roadway settlement and piping in many areas where roads cross or follow arroyos in the area (Parker and Jenne, 1967).

Local geotechnical consulting firms and the USBR have shown that these soils are highly susceptible to collapse. Consolidation testing of some sandy-silty clay samples (with densities as low as 80 pcf) revealed wetting-induced collapse of more than 12 percent at

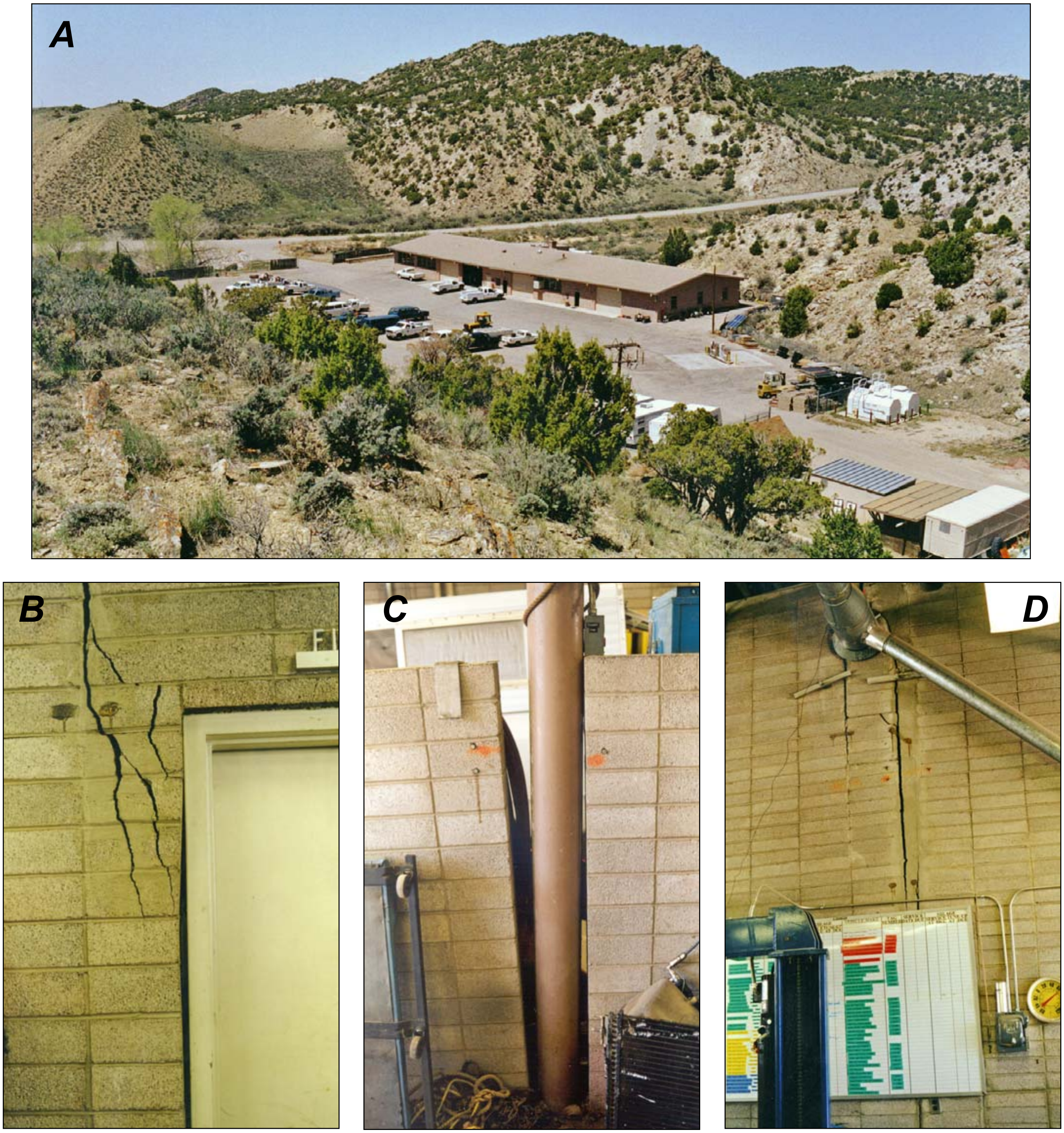


Figure 7-18. The Dinosaur National Park maintenance facility is nestled within sandstone ridges (hogbacks) formed by the tilted Frontier Member of the Mancos Shale, which marks the southern edge of the Uinta uplift. *A*, Photo taken from atop the first hogback ridge, looking across (west) to floor of Dripping Rock Creek and succession of ridges beyond in left-center background. Note the proximity of the right side of the building to the whitish-gray sandstone ridge where bedrock below the footing is shallow. On the left side near the valley floor, thicker soils are present, including the alluvium of Dripping Rock Creek and colluvial soils from the ridges. *B–D*, Brick distress and tilting near a central support post of the structure, resulting from several inches of differential settlement of the building side that is underlain by the thick, collapse-prone soil. In C and D, the orange spots are paint marks that showed the locations of crack gages to monitor the structure movement.

1,000 psf; rapid compression continued (i.e., a very steep consolidation curve) upon higher incremental loads. The USBR reported mudflow alluvial-fan deposits as thick as 80 ft, indicating that large magnitudes of ground settlement could be possible in these areas if deep wetting occurred (Luehring, 1988). The newer Ute Mountain Casino on U.S. Highway 160, where these problematic soils exist, was placed on end-bearing piles, driven through the 30 ft of collapse-susceptible soil into the underlying shale bedrock (Ben Cordova, personal commun., 1997).

The soils of the Four Corners area have been mapped by soil scientists from the NRCS field office in Cortez, who recognized that certain soils are highly gypsiferous (Doug Ramsey, personal commun., 2001). The resulting soil survey of the Ute Mountain Reservation has informative tables with bulk density, percent gypsum, and sodium-absorption ratios (SARs) for each mapped soil unit so that rough assessments can be made on collapse, susceptibility to dispersion, and piping erosion.

8. GEOLOGICAL AND GEOTECHNICAL INVESTIGATIONS FOR COLLAPSIBLE SOILS

Residential home construction is particularly vulnerable to distress or damage from ground settlement due to soil collapse because the developer, homeowner, or builder might commission only limited investigations and engineering to reduce costs. They may also choose to ignore recommendations and take risks with inadequate designs because they are not fully cognizant of the hazards of adverse wetting of potentially collapsible soils and the risk of damage that could result (Houston and Houston, 1989). For building purposes, it is important that a site investigation be performed to determine how the specific conditions could affect the proposed development. An investigation is necessary not only for the characterization of collapsible soils or if collapsible soils are suspected, but for problem soils, in general. This chapter discusses the different types of reports that could be prepared for a site, how they could be used, what should be emphasized in technical reports covering a site known to have collapsible soils, and what should alert a homeowner or prospective buyer.

ENGINEERING GEOLOGY OR GEOLOGIC HAZARDS REPORT

The geology report provides the baseline information and interpretations for a particular site that will point to the proper level of further geotechnical investigations. The geology report may be supplied by the seller; reports might also be filed at the county or municipal planning or building office and at the Colorado State Archives, which stores the CGS land-use files.

There are certain guidelines for any geologic report that discusses geologic hazards (Rogers and others, 1974; Shelton and Prouty, 1979), but the geologic report should further emphasize the following points for those areas that may be considered susceptible to collapsible soils:

1. A description of the local bedrock is vital. Are the silt and clay contents of the bedrock high? Is it poorly indurated and easily erodible?

2. Is there poor plant growth that would indicate accelerated erosion? Does the terrain resemble badlands? What about high salt and sulfate contents?

3. An interpretation of the local geomorphology and geologic history of the formation of the surficial sediments (soils) should be included. This section should cover the type of sediments, the source rock, how the sediments were deposited, when they were deposited, and so forth. Chapter 5 discussed how certain landforms and sediment depositional systems lead to formation of soil with collapse potential.

4. The climate of the area is critical. What is the average annual precipitation at the site?

5. A geologic map and a geologic constraints map should be constructed and included in the report. The mapping must be completed prior to the laying out or "cookie cutting" of residential lots on development plans.

The geology report must be prepared by a professional geologist (as defined in Colorado Revised Statutes, 2007). Currently Colorado does not require licensure or registration of geologists, but Colorado Revised Statutes (CRS) do require that geologic reports, or the geologic content of other reports, be prepared or authorized by a professional geologist, as defined in CRS 34-1-201. Where the geologic and geomorphologic conditions suggest that collapsible soil is present, a certain level of geotechnical investigation is warranted, and site-specific foundation investigations are needed.

GEOTECHNICAL ENGINEERING INVESTIGATION

The geotechnical engineering study generally includes a subsurface soil investigation that characterizes the soil conditions for engineering purposes. A geotechnical report for a site might be on file with the builder, the local planning office, or the State.

In the subsurface investigation, samples are collected from a borehole or a test pit and analyzed in a soil laboratory for different soil engineering properties. These analyses are used in the design of pavements, foundations, retaining walls and other ground modifications, slope and excavation stability, and individual sewage disposal systems (ISDS). This work must be completed under the supervision of a licensed professional geotechnical engineer.

One of the most significant parameters to quantify during a subsurface investigation is the thickness of the collapsible soil. At least one boring in the subsurface investigation should be advanced through the entire collapsible-soil column, unless the geotechnical consultant has experience in the immediate area and has good control for determining either (1) the depth to a noncollapsible-soil substrate or bedrock or (2) the depth at which the soil has been saturated and collapse cannot occur upon further wetting. Many residential investigations evaluate only the subsurface conditions for an already-assumed shallow foundation; thus, the termination of the boring or shallow test pit approximates the depth where the influence of the stress-distribution envelope (or bulb) of the foundation is judged to become insignificant at the assumed bearing load. Even low collapse potential for thicker layers of collapsible soils may still present problems if the entire soil becomes wetted (e.g., 30 ft of soil having 1 percent collapse potential could still settle 3.6 in. if the soil column becomes fully saturated under load). Without knowing the thickness of a collapse-prone soil, the geotechnical engineer cannot reliably determine the potential severity of the settlement risk.

The geotechnical methodology needs to vary with the type of collapsible soil that is present. As was discussed in Chapter 4, collapsible soils can react differently, depending on the formational source. More-granular soils can have very high rates of immediate mechanical collapse upon wetting, and upon additional loading, the result is a flatter postsaturation consolidation curve (see fig. 4-3). Gypsiferous soils might have little or no collapse upon loading, but could show long-term settlement potential or be highly altered where previously irrigated. Other, more cohesive, denser, clay-rich soils, such as the fine-mud alluvium derived from the Mancos Shale, might have little immediate collapse or slight expansion upon wetting at low loads, but very steep consolidation curves upon additional loading when saturated. In highly variable soils, sampling every 5 ft might miss the soil layers with the highest collapse potential.

In areas where dispersive soils and piping voids are known to exist, it should be determined whether they are present below a proposed structure footprint or road or utility alignment. Structures should not be sited in potentially erodible soils near a steep-walled arroyo unless extensive subgrade soil modifications are completed. There are various geophysical methods that can identify shallow subsurface voids. Ground-penetrating radar (GPR) has been used with reported success by Werle and Stilley (1991) in Nevada. Seismic and electric resistivity surveys might also distinguish voids in the subsurface.

The geotechnical report should also provide drainage recommendations. Drainage and water-management design criteria are extremely important, and a geotechnical report in collapse-prone areas would be inadequate if it did not offer site-specific drainage recommendations. It is often 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. Utility trenches need to be properly backfilled and compacted to prevent the formation of inadvertent pathways of moisture migration. Site grading should provide positive slopes for structures. Grading plans should take care not to alter the natural surface drainage of the slope without providing compensating improvements. For example, the construction of a driveway embankment over a shallow swale can impede drainage and result in temporary pooling of water during heavy rainfall. This area could be a site of subsequent collapse and threaten nearby structures. Drainage concepts are covered in Chapter 10.

FOUNDATION REPORT

A foundation report may be supplied by the builder as a requirement for a building permit and should be on file at the local building department. This report includes the structural design parameters for a building. Ideally, a critical analysis would have been made of the home site in relation to the landscape, and the home would have been designed by using soil-test data and the conclusions of the geotechnical report. The foundation report should be read by the prospective homeowner, and a general acceptance of the report should be a condition in the real estate purchase. It is the responsibility of the homeowner to speak with the geotechnical engineer regarding the results of the investigation and the foundation recommendations. Homeowners who have doubts or further concerns based on the content of this publication should obtain a second opinion from a qualified third party.

For older homes, there may be no geologic or subsurface geotechnical investigation available. In those cases, the content and maps referenced in this publication and NRCS soil surveys may be helpful. The prospective homeowner could retain a professional geologist or geotechnical firm to conduct the recommended background research.

MAP RESOURCES

Quality geologic maps and soil maps are available for large parts of Colorado. Geologic maps for an area can be found through the USGS National Geologic Map Database at the Web site http://ngmdb.usgs.gov/. CGS will also soon have on their Web site the House Bill 1041 geologic hazards and constraints maps that were created by both consulting and State geologists for many of the mountainous and western counties of Colorado. Several maps specifically dealing with geologic hazards and collapsible soils are available from the CGS; most of these were discussed in Chapter 6.

There are NRCS soil maps for most counties of Colorado. The NRCS maps have been, or are in the process of being, digitized and can be downloaded as GIS files from the NRCS Web site at **http://soildatamart.nrcs.usda.gov/**. Most of the soil surveys have been completed for primarily agricultural purposes, but many contain measured index properties and engineering application tables that list levels of severity for various types of engineered works. Other useful online resources include the NRCS Web pages and both the publications *National Soil Survey Handbook* and *Understanding Soil Risks and Hazards* (NRCS, 2007; Muckel, 2004), which contain a wealth of information on soil formation, properties, and hazards.

9. MITIGATION TECHNIQUES FOR COLLAPSIBLE SOILS

Many different methods are used for mitigation of collapsible soils. Many are the same methods employed for construction of foundations on soft or unstable ground that is not necessarily collapsible. Some methods are the same as those recommended for swelling (expansive) clay. Others are techniques that have been developed specifically for collapsible soils. The material in this chapter has been compiled from many sources, but primarily from Clemence and Finbarr (1981), Luehring (1988), Houston and Houston (1989, 1997), Pengelly and others (1997), Rollins and Rogers (1994), and the commercial literature of Hayward Baker. The specific mitigation techniques can be divided into four basic groups: avoidance of collapse-susceptible soils and (or) adverse wetting, ground modifications that mitigate or remove soil-collapse potential, structural reinforcement techniques, and deep foundations to transfer loads.

The specific mitigation technique in each of these groups is discussed in the following sections. The best mitigation might incorporate techniques from more than one group for a final engineering design. At the end of this chapter is a discussion of remedial techniques commonly used to repair structures that are threatened or have been damaged by collapsible-soil settlement

AVOIDANCE OF COLLAPSE-PRONE SOIL AND ADVERSE WETTING

With advance planning and sufficient available land, it may be possible to avoid those areas with appreciable thicknesses of collapse-susceptible sediments and to site structures on less problematic terrains. For example, on slopes that flatten toward river terraces, moving downslope could result in a thinner section of collapse-prone soils overlying densely packed river gravel, which is generally an excellent foundation material. Likewise, farther up the slope, there might be a location where the foundation could be placed on more favorable pediment gravel or even bedrock.

The avoidance of wetting is more difficult to achieve and cannot be assured. The wetting of subsurface collapse-prone soils should be avoided or reduced as much as possible, regardless of the mitigation technique selected (unless the mitigation option is to prewet the entire soil depth and induce collapse, a ground-modification technique that is discussed later in this chapter). Simple avoidance of wetting is not recommended as a stand-alone mitigation for collapsible soils and is not generally considered good engineering practice.

Chapter 10 discusses drainage recommendations that would reduce potentially adverse wetting and improve long-term foundation performance. However, accidents occur. Water and sewer lines leak or break. Poor grading and landscaping design, irrigation, and inherent moisture increase by capillary action in subsoils under impermeable slabs may increase the degree of saturation to the threshold where collapse of the soil structure could occur. The probability of wetting at some point in the normal lifetime of a structure, 50 to 100 yr, is typically high (Houston and Houston, 1989). Most geotechnical firms in Colorado provide specific recommendations to reduce the wetting potential of collapsible soils, regardless of the foundation type or mitigation method(s) chosen.

GROUND MODIFICATIONS TO MITIGATE OR REDUCE SOIL-COLLAPSE POTENTIAL

There are several ground-modification techniques to mitigate and (or) reduce the collapse potential of susceptible soils. Although the methods vary considerably, the common purpose is that the ground is modified, generally by densification of the collapsible soil (i.e., removing void space) or strengthening (further cementing) the metastable binding agents.

Removal of Moisture-Sensitive Soil (Overexcavation)

A deposit of collapse-prone soil that is relatively thin could be removed by grading or foundation excavation. Geotechnical engineers should always consider this approach because of the relatively inexpensive cost. In some cases, the inclusion of a basement that was not initially considered or desired by the client could result in excavating the entire collapsible-soil column, and the footings would then bear on a more suitable substrate, such as bedrock or river-terrace gravel (**figure 9-1**).

Overexcavation and Replacement

The method of overexcavation and replacement is now extensively used in Colorado for residential home construction and is effective for shallow foundations in both collapsible soils and expansive soils, provided there is good construction control. A suitable thickness of soil is excavated below the footing elevation and replaced with engineered fill that is moisture conditioned and compacted in a succession of thin soil layers called “lifts” to a prescribed moisture and density (**figure 9-2**). The method of overexcavation and replacement has also been utilized to verify and (or) prevent the formation of piping voids and

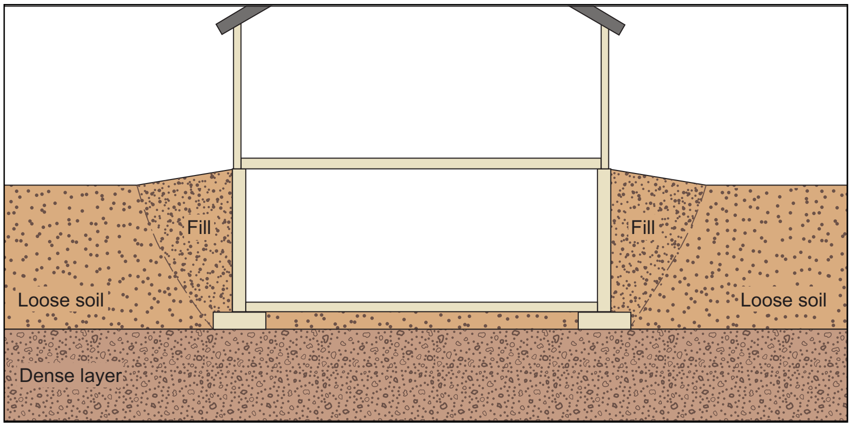


Figure 9-1. A basement excavation through collapsible soil to a denser material below. Note the proper compaction of backfill and positive grading away from the structure walls.

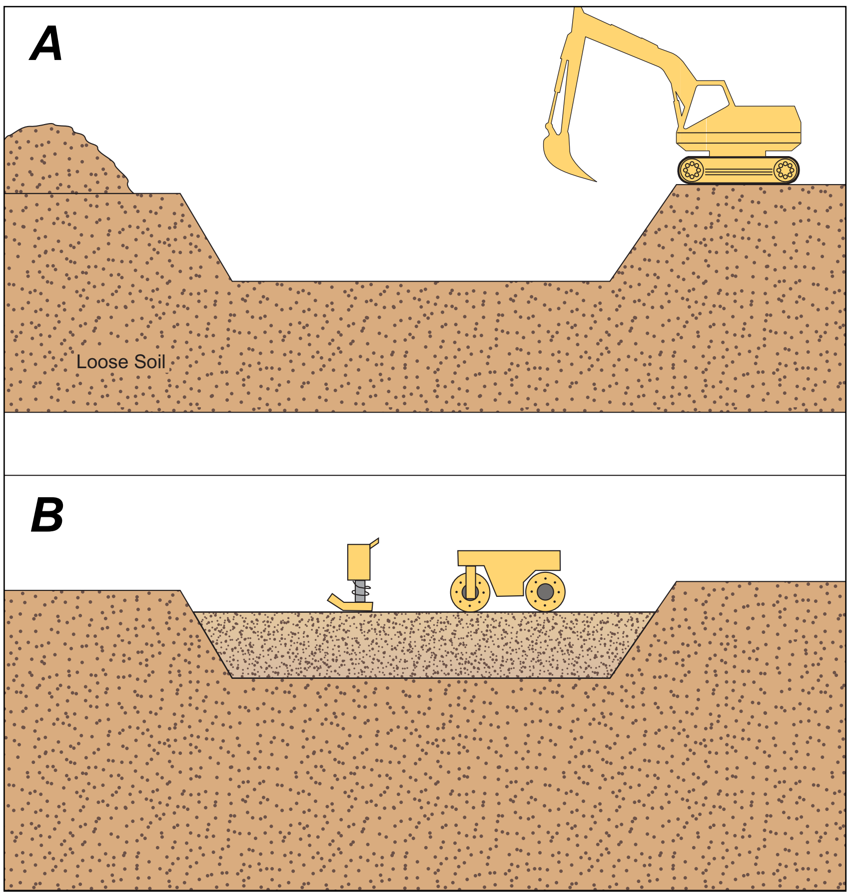


Figure 9-2. Overexcavation.
***A*, Collapsible soils are removed to a predetermined depth.**

***B*, Replacing the collapsible soil is engineered fill with a prescribed moisture content and compacted density.**

pseudokarst phenomena within an area of concern. With hydrocompactive soils, especially those with a significant gravel and sand component, generally the same soils that were excavated may be used for the fill. Such soils—when compacted and moisture conditioned—provide both excellent bearing characteristics and a lower-permeability "soil mat" for the shallow foundation

Importing coarse or well-sorted (i.e., poorly graded) granular soils is generally not recommended because of the cost and also because the high permeability could create a subsurface "bathtub" effect in which moisture and ground water actually collect and pond within the fill. This perched water in the fill could then slowly infiltrate into the underlying and adjacent collapsible soils. For imported fill, a select granular material is recommended, such as a well-graded ¾-in. road base, which will be relatively tight when compacted and will have a permeability similar to the native soils. If the native soils are very clayey, then import of more granular fill might be necessary to prevent expansion (swell) potential of the fill. Geotextile reinforcement can be added to increase the rigidity and bearing capacity of the substrate and provide separation between the backfill and underlying fine-grained soils.

In addition to partly removing the collapsible soil at the building site, the method of overexcavation and replacement creates a stronger soil mat that distributes bearing-load stresses from above and potential settlement strain from below. This type of mitigation would not prevent settlement if the column of collapse-prone soil is so thick that a part of it still remains below the bottom elevation of the reinforced soil mat. Upon deep-seated wetting, surface settlement could still occur, but the rigidity of the soil mat would distribute the strain so that the differential settlement would be reduced and less distress and damage would occur to the structure.

Prewetting

Prewetting is a method whereby the soils are intentionally saturated, and soil collapse and ground settlement are induced prior to development. This saturation can be induced by sprinkler irrigation, flood irrigation, trenching, ponding of water, and well injection. Commonly, settlement can be observed by the soil cracks and ground fissures that mark the advancing front of subsurface wetting.

Prewetting was one of the first mitigation methods for orchard fields in the arid Western States (Paddock and Whipple, 1910). Geologists and engineers with the USBR published their experience with prewetting for irrigation canals in the central valley of California and in southwest Colorado (Gibbs and Bara, 1962, 1967; Bara, 1975; Prokopovich, 1984; Luehring, 1988). Prewetting has also been performed in collapsible-soils research to evaluate collapse potential and to do comparison testing of other mitigation options, with and without prewetting, in New Mexico (Shaw and Johnpeer, 1985b; Reimers, 1986), Utah (Rollins and Rogers, 1994), and Colorado (Shelton and others, 1977).

This method has been used in Colorado, albeit rarely, for some residential locations. In Meeker, a 4-ft-deep foundation excavation for a three-story brick-faced apartment building in a highly collapse-prone part of town was completely filled with water that was allowed to soak into the ground. The wetted excavation floor was then compacted with a vibratory roller compactor before shallow spread footings were constructed. The apartment building has shown only minor settlement distress, whereas a similar structure on an adjacent property, also founded on shallow footings but without prewetted soils, was subsequently condemned and demolished because of the damage from differential settlement (Jim Komotinski, personal commun., 1997).

CDOT used prewetting to induce settlement of collapsible soils on large alluvial fans along the Interstate 70 alignment west of Rifle (figs. 4-4 and 6-2). The results were mixed. Settlement continues on the roadway where thick fills were placed on collapsible alluvial-fan soils and drainageways were altered. CDOT therefore performs continual maintenance related to roadway distress and settlement from soil collapse on Interstate 70 (Alan Hotchkiss, personal commun., 2001).

Prewetting, as a stand-alone mitigation technique, has significant limitations. There can be uncertainties as to whether the wetting front is saturating all the soils uniformly and whether there is vertical penetration into lower-permeability soil layers. Experience has shown that wetting fronts in very dry, clay soils can be highly irregular and seemingly random (El-Ehwany and Houston, 1990). In forensic geotechnical investigations of ground settlements due to hydrocompaction and hydrocompression, bone-dry areas of soil have been reported adjacent to fully saturated zones (E.O. [Ed] Church, personal commun., 1998). To ensure that an entire column of collapsible soil is properly saturated to a prescribed lateral distance and vertical depth, costly infiltration holes would need to be drilled or hollow probes would need to be advanced into the problem soil needing treatment. Also, unless additional mechanical compaction is performed, the induced collapse of the soils would occur only from the existing saturated-soil overburden stresses (i.e., the weight of the wet soil). Prewetting can be useful for roadways and irrigation canals where the induced loads other than overburden stresses are small, but prewetting is not generally recommended as a stand-alone mitigation to prevent future distress of any type of engineered foundation that would impose higher load stresses on the soil. Rollins and Rogers (1994) showed in their foundation testing that prewetting without additional loading on the collapse-prone soil does not result in substantial decrease of void space, and later loading could still cause significant settlement. This concept is important in Colorado because of the misconception by some that riverside alluvial-fan soils in Colorado that have been flood irrigated for farming and ranching are no longer prone to collapse.

For engineered foundations, prewetting techniques are most effective when utilized with other techniques, such as surcharge loading or postwetting vibratory compaction, which are discussed in the next section. Prewetting is also not suitable for very clayey soils that may not collapse unless loaded or that become hydrocompressible and require longer periods of consolidation time. Infiltration wells or probe injection at a predetermined spacing would ensure that complete saturation of the collapsible-soil column will occur.

Compaction by Vibratory Plate, Vibratory Roller, and Tamper, Combined with Prewetting

Compaction of soil using typical heavy equipment, such as sheepfoot compactors, vibratory roll compactors, vibroplate compactors, and tamping rammers (e.g., Wacker Packers™) may be used to induce collapse of near-surface collapsible soils. Soils are generally pretreated, in situ, by deep plowing or ripping to break them up and make them more permeable. It is recommended that collapsible soil be moisture conditioned or prewetted prior to compaction so that the inherent strength of the soil in the dry state would be removed and the vibration or static impact energy could then densify the soil. This method would only mechanically compact the soil to the depth rated for the particular compaction device and to the depth where prewetting has achieved sufficient saturation to loosen soil-binding agents and induce collapse. This specified compacted depth can often be relatively shallow and might be insufficient to prevent significant settlement for thicker collapsible-soil layers. The work by Bennett (1983) indicated that the combination of deep plowing, wetting, and surface vibratory compaction was the least effective of the methods tested by New Mexico Department of Transportation and was considered unacceptable for thick collapsible-soil deposits.

Deep Blasting Combined with Prewetting

The relatively instantaneous ground acceleration or seismic shock wave from deep blasting of prewetted collapsible soils may break down the binding structure of the soil so that it would densify under its own weight. Russian researchers have developed two techniques for this approach (Houston and Houston, 1989). One method uses a single deep blast, and the other uses a set of simultaneous blast charges in several boreholes. The subsidence and blast cavities are subsequently filled from the surface with water and then with sand and gravel.

This approach was tested by CDOT Region 3 during the Interstate 70 research and construction through collapsible-soil terrain west of Rifle. In an infamous episode, still remembered by CDOT personnel, the test blast hole was loaded “hot” (more explosives than needed), and many CDOT vehicles and personnel were splattered with ejected, prewetted, collapsible soils (i.e., mud). The project engineer was known thereafter as “Boom-Boom” (B. Barrett, personal commun., 2004; R. Perske, personal commun., 2003) The best approach for this method would be to use smaller explosive charges at tighter spacing with timing delays.

Chemical Stabilization

Chemical stabilization or permeation grouting techniques are used to introduce a solution into soil pores and provide additional cementation to the soil, which strengthens its framework to prevent or reduce settlement. This technique only partially reduces void space, but the cementation strengthens and solidifies the soil-particle contacts. The chemical stabilizers include sodium silicate solutions, gaseous silicatization, ammonia, polymers, or a silicate grout.

The published record mostly discusses the sodium silicate solution, in which the silicate combines with salts in the soil to form an insoluble, cementitious gel that coats the soil grains, further binding them (Houston and Houston, 1989). Russian researchers have pioneered the bulk of this work and have used sodium silicate strengthening in dry and wet loess soils in the former Soviet Union and Bulgaria. Although the sodium silicate solution has been used mostly outside of the United States for mitigation of loess soils, Rollins and Rogers (1994) found this type of chemical stabilization useful in their testing of mitigation methods in alluvial-fan soils of Utah. Their analysis showed that for two equivalent test cells of soil, one injected with sodium silicate solution and one injected with an equal volume of only water, there was 70 percent less collapse with the chemical stabilization. The results suggest that the cementing action is very rapid, which could make sodium silicate wetting or injection a valid remedial mitigation method when settlement is first noted or suspected (Rollins and Rogers, 1994). A 5 to 20 percent sodium silicate solution is recommended; it either infiltrates or is injected into the soil similar to the way prewetting is commonly performed (Rollins and others, 2002). Chemical stabilization is a relatively expensive method and is useful for treating both collapsible and swelling soils. Hayward Baker, a firm that specializes in ground modifications, has an injection system mounted to a bulldozer that can inject as many as five probes simultaneously in a predetermined grid to treat an entire swath of ground in one pass (**figure 9-3**). In addition to cementing agents and slurries, this system can also inject water (i.e., a prewetting mitigation technique). A limitation of this system could be the difficulty in advancing the probes through hard soil.

In situ treatments of cohesive, clay-rich soils by chemical grouts are not generally recommended because of the difficulties inherent to the limited permeability of the soil. Chemical stabilization can also include soil-additive treatments designed to be used during excavation, grading, and structural fill placements. For dispersive clay soils and piping alluvial soils, Sherard and others (1976b) found that,

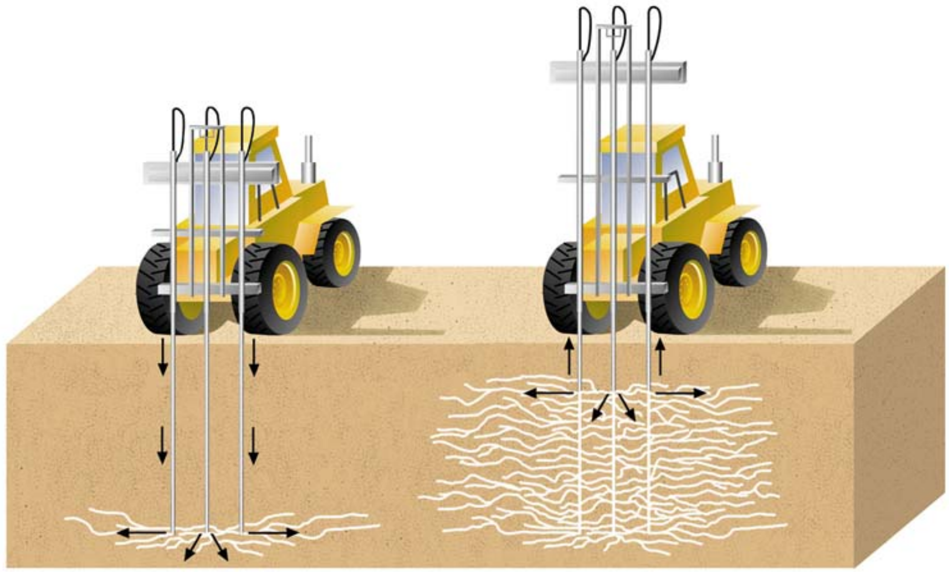


Figure 9-3. An injection probe array mounted to a bulldozer. Image courtesy of Hayward Baker and used by permission.

without exception, clay soil material that was treated with lime—$Ca(OH)_2$—was transformed to a nondispersive state. Also, adding small percentages (on a dry weight basis) of alum, fly ash, agricultural lime, and magnesium chloride can transform dispersive clay soil to a nondispersive state (NRCS, 1991).

A phenomenon to consider when discussing lime treatments is the role of gypsum in the deformation of lime-stabilized soils. In a moist lime-treated soil or fill, gypsum can supply sulfate for the growth of the expansive sulfate mineral ettringite ($Ca_6Al_2(SO_4)_3(OH)_{12}\ 26H_2O$), which can cause the treated soil to swell and heave (Burkart and others, 1999). Ettringite has caused significant deformation of pavements in central Texas, where Cretaceous shales and derived soils with high gypsum content are similar to soils of western Colorado. For treated soils, the potential of this reaction should be considered. The expansion during the crystallization of ettringite is the primary cause of corrosive damage to concrete that is not sulfate resistant.

Compaction Grouting

Compaction grouting is a common remediation for all types of structures that have experienced distress in collapsible-soil terrain in Colorado. Unlike chemical stabilization, compaction grout does not enter the soil pores, but displaces the soil and densifies it. There may be a slight cementation component. The method consists of drilling a boring to a prescribed depth, wetting the drilled soil column, and injecting a low-slump cementitious grout under suitable pressure to the point where the native soil is displaced. The injected grout grows as a "bulb," producing controlled radial displacement of the soil as it accommodates the volume of grout and as densification of the soil occurs (Marin and Von Fange, 1993). Compaction-grouting technologies using expansive, polyurethane foam have recently entered the marketplace.

There are two general approaches to compaction grouting. The first is used for preliminary mitigation of a development site known to have extensive collapsible soils. A grid of grout injection borings is laid out to achieve good densification of a wide area through creation of a thick mat of dense subsurface soil prior to excavation and foundation construction. In the other approach, a subsurface grout column is injected to support a structure founded in a thick section of collapsible soils. For this method a boring is drilled though the footing or immediately adjacent to a drilled shaft and advanced to a suitable bearing substrate below the collapsible-soil layer. A grout probe is inserted to the bottom of the hole, and a series of grout bulbs are injected into the soil under pressure in a bottom-up direction, one on top of the other, extending upward to the footing elevation or tip elevation of the deep foundation (**figure 9-4**). Essentially, the grout column becomes a deep foundation that then supports the structure's original foundation.

Good control and inspection are needed to ensure grout effectiveness and consistency in dry collapsible soils. It is important to confirm that the compacted grout column extends through the problem soil to a suitable bearing material. If the bottom of the grout column remains within the collapsible soil, continued wetting and further ground subsidence may cause the grout column to settle in the future (Steve Pawlak, personal commun., 2005).

Compaction-grouting remedial repair has been used at school and municipal facilities in Rangely, facilities at the Dinosaur National Park headquarters, the Montrose County court house, the Glenwood

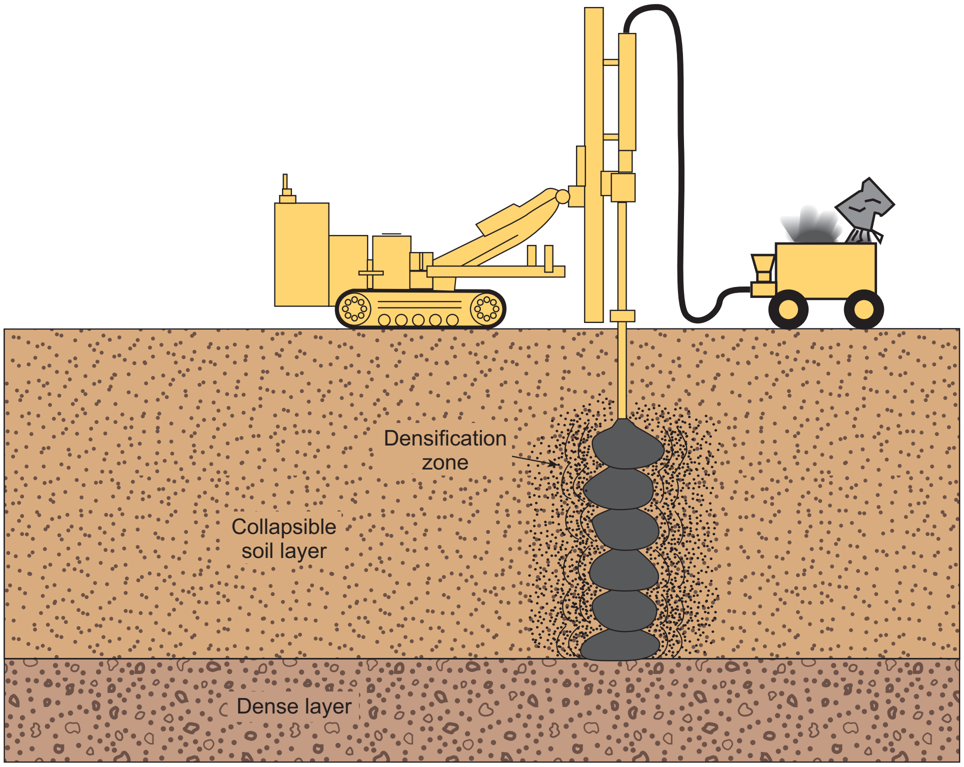


Figure 9-4. Compaction-grouting treatment, in which grout is injected under pressure, displacing the soil and making it denser.

Springs water treatment plant, and countless residential buildings in Colorado, all of which had settled because of the wetting of collapsible soils. Compaction grouting is an excellent, although costly, mitigation solution to prevent or greatly reduce additional settlement in the future.

Compaction grouting should not be confused with mudjacking. Mudjacking, also called slab-jacking, is the injection of mud (that usually contains some cementitious grout) under a concrete slab to relevel it. It does nothing to modify the collapsible nature of the soil. In fact, while mudjacking in collapsible soils might relevel the slab in the immediate short term, it could actually accelerate settlement because overburden stresses have increased from the additional load or weight of the mud. Further remediation could be required in the future. If the section of collapsible soil is thin and fully wetted such that all collapse potential has been removed, mudjacking would be effective for the long term.

Dynamic Compaction

Dynamic compaction is a method of increasing soils' density by using a modified crane to repeatedly drop a heavy weight, or tamper, from a prescribed height. The tamper can weigh 10 to 20 tons and may be dropped from as high as 100 ft. The tamper is dropped several times on the same spot, and the shock of the impact propagates a cone-shaped wave of energy that breaks the soil-grain contacts and makes the soil denser (**figure 9-5**). Compaction and volume loss is easily seen by the craters that are formed from the impacts (**figure 9-6**). The crater surfaces are subsequently leveled out and moisture conditioned during subgrade compaction as part of site grading. Improvement to depths of 20 to 25 ft can be achieved with equipment that is typically available.

An evaluation of collapsible-soil mitigation methods published by the New Mexico Department of Transportation (NMDOT) concluded that dynamic compaction was the most effective method, but was by far the most expensive (Lovelace and others, 1982; Bennett, 1983). The Federal Highway Administration recognized the validity of this type of ground modification and published guidelines on dynamic compaction for U.S. highways (Lukas, 1986). On the basis of the results of the earlier study, NMDOT utilized deep dynamic compaction on 30-ft-thick collapsible-soil deposits on Interstate 25 north of Albuquerque and on Interstate 40 west of Rio Puerco (Pengelly and others, 1997). Kyle M. Rollins of Brigham Young University has conducted much research on dynamic compaction and found it to be the most effective of all ground modifications in mitigation of collapse potential (Rollins and Rogers, 1994; Rollins and others, 2002). The Wyoming Department of Transportation (WyDOT) has specified mitigation by dynamic compaction on new road and rehabilitation projects in areas of low-density, collapse-prone soils since the mid-1990s and

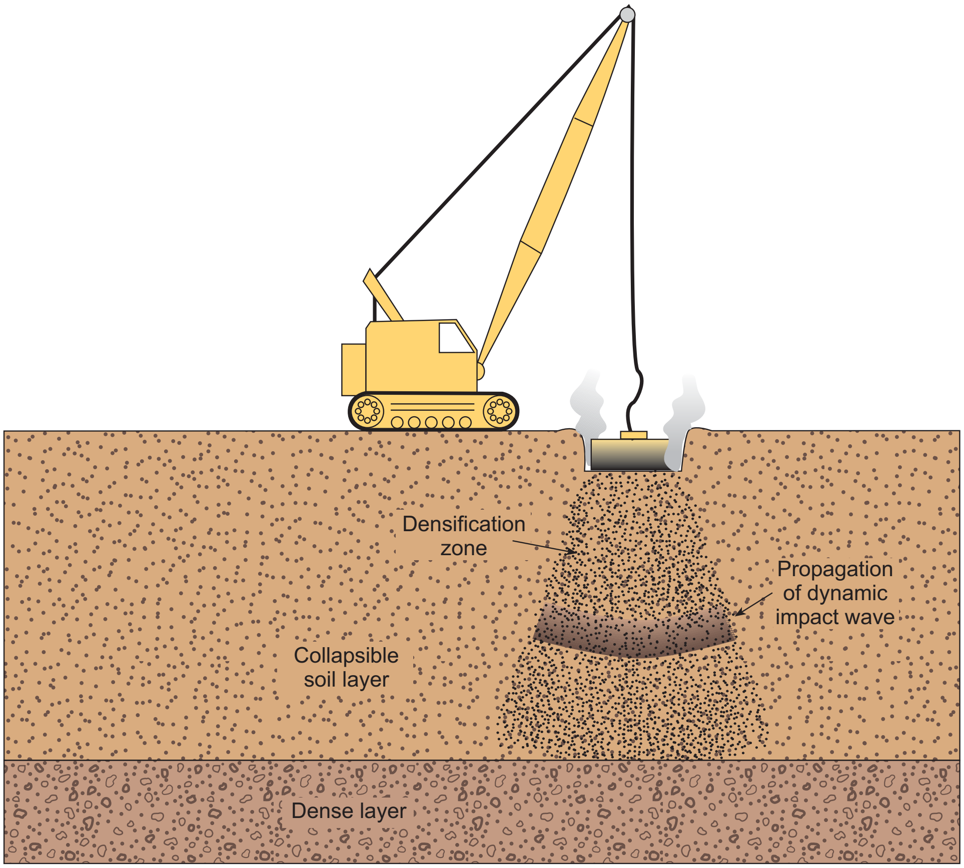


Figure 9-5. Dynamic compaction. Densification and volume reduction of the collapsible soil from the dropped weight will leave a crater at the surface.

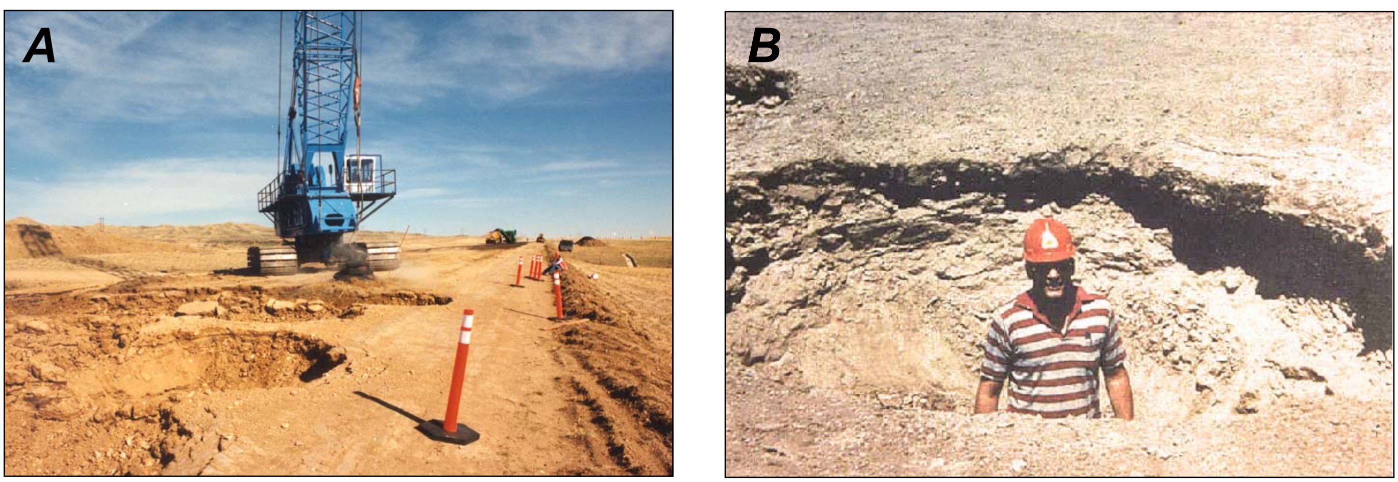


Figure 9-6. Displacement and cratering of the ground surface during a dynamic compaction project in Wyoming. *A*, A modified crane dropping a weight has created a crater grid along the road alignment. *B*, The depth of the crater that can result from the dynamic collapse and densification of the low-density soil. For scale, man standing in crater is 6 ft, 4 in. tall. Photos courtesy of Mike Hager and Wyoming Department of Transportation.

has written special provisions to WyDOT Standard Specifications for its use (Michael Hager, personal commun., 2001). CDOT has used dynamic compaction only in soft, former landfill locations in the Denver metro area.

Dynamic compaction is best used in classic CL-ML, low-plasticity, low-density hydrocompactive soils. Work by Rollins (personal commun., 2006) indicates that high clay content in soils, or the presence of clay layers within the zone of the energy wave, will attenuate the impact energy and lessen its effectiveness to increase the soil density below the clay.

Compaction by Displacement Piles

For this procedure, piles are driven through the collapsible-soil layer and then withdrawn. The hole is then filled with lifts of soil and (or) rock that is compacted in lifts by a tamping hammer to create a compacted fill column within the low-density soil layer. This concept is similar to bottom-up compaction grouting, whereby a column of denser compacted soil is used to transfer foundation loads to stiffer soil or rock below the collapsible-soil layer. Because the collapse potential of soils between the intervening piles has not been affected, foundations must be designed with grade beams to span the distance between the displacement piles.

Vibro-Compaction, Stone-Column Replacement, and Jet Grouting

The techniques of vibro-compaction, stone-column replacement, and jet grouting are advancements of the displacement-pile technique just described. In these techniques, a column of reinforced soil, grout, or rock is placed through a collapsible-soil layer. Vibro-compaction is a process in which a vibrating probe that jets water and compressed air is advanced through a soft soil layer. As the vibrating probe is removed, the column of wetted, granular soil becomes denser and is reinforced. In cohesive, clay-rich-soil applications, graded rock is also dumped alongside the vibrating probe as it is slowly withdrawn. The result is a compacted column of rock that has been placed through the soft soil layer (**figure 9-7**). The benefits of this process are (1) there is a deeper wetting of the soil by the advancing jetting probe, (2) compaction occurs by vibration, and (3) replacement of the native soil with a rigid stone column will partially behave as a deep foundation and transfer load to the more suitable strata below the collapsible-soil layer.

Experimental compaction of collapsible soils by vibro-compacted stone columns was conducted by NMDOT in 1981 (Bennett, 1983). The conclusions reached were that this method produced more improvement to the collapsible soil than prewetting or surface conditioning and compaction, but wasn't as effective as dynamic compaction. Notable concerns were the potential for future collapse between the vibro-compacted columns, especially where the probe spacings were farther apart (e.g., 9 to 12 ft), and the soil-moisture content and density did not appreciably change. This limitation may be overcome by designing grade beams to span the distance between columns, similar to the procedure for displacement piles. The stone columns may also provide a pathway for water infiltration from the surface and lead to future wetting of the adjacent, still-collapse-prone soil (Kyle M. Rollins, personal commun., 2006). For these reasons, caution is needed for use of this mitigation technique.

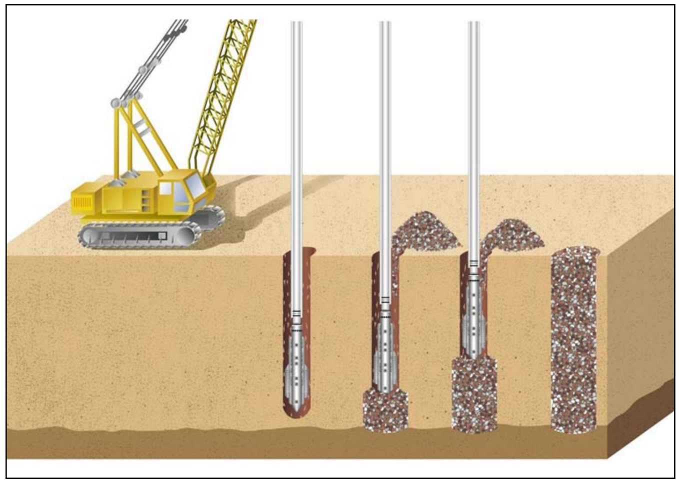


Figure 9-7. Vibro-compaction and stone-column process. Image courtesy of Hayward Baker and used by permission.

It is unknown whether vibro-compaction has been used in Colorado specifically for mitigation of collapsible soils.

In the jet grouting technique, the probe or drill steel does not vibrate, but the jetting water and compressed air mixture causes the surrounding soil to saturate and erode. After advancing down to an acceptably dense substrate, the drill pipe is withdrawn while grout is pumped under pressure through horizontal nozzles to mix with the wetted and disturbed soils (**figure 9-8**). The resulting subsurface grout and soil mixture then cures and forms a hardened, columnar, structural element similar to the stone column previously described. Jet grouting does not appreciably increase the density of the surrounding native soil, so foundations would also need to be designed with grade beams to span between the grouted columns (Marin and Von Fange, 1993)

Other Unproven or Potential Methods for the Future

Other methods proposed in the literature bear mentioning, although they either have been used only for pilot studies or have only been hypothesized. The first is thermal alteration to solidify collapse-prone soils (Clemence and Finbarr, 1981). Russian researchers used thermal and thermochemical processes to change the stability of loess (Luehring, 1988). This process bakes, melts, or vitrifies the soil to fuse or weld the soil-binding agents so that collapse cannot physically occur under normal loading conditions, regardless of additional moisture that may be introduced. Different process methods include the circulation of superheated air into a boring, the downhole injection and ignition of highly combustible products that burn at very high temperatures, and downhole use of a specialized arc-welding torch.

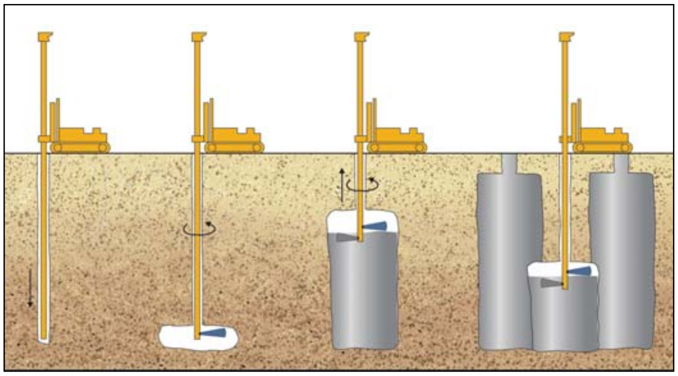


Figure 9-8. Jet grouting procedure. The blue jet shown is water, and the gray jet is the grout that mixes with the soil to form the gray grout columns shown. Image courtesy of Hayward Baker and used by permission.

Another possible method is ultrasonic vibration that could destroy the binding mechanisms of the metastable fabric of the soil, inducing collapse and densification. We found no published references on the use of this method.

STRUCTURAL AND STRUCTURAL REINFORCEMENT TECHNIQUES

There are different structural solutions to reinforce shallow, engineered foundations to accommodate the strains that could result from differential settlement after soil collapse. The systems recommended to mitigate the effects of collapsible soils are differential settlement–resistant foundations that bridge or cantilever over potential soft zones (Clemence and Finbarr, 1981; Houston and Houston, 1989). The major systems include reinforced continuous-footing designs for spread footings, grid footings instead of isolated internal bearing pads, reinforced mat foundations, and post-tensioned slabs.

Reinforced spread-footing walls are commonly used in Colorado in soils with slight to sometimes moderate collapse potential. The reinforcement adds two and sometimes four longitudinal, steel reinforcement bars (rebars) to stiffen the concrete foundation wall. This reinforcement acts as grade beams and enables the foundation stem walls to span 10 to 14 ft of differential ground settlement without cracking or undergoing some other form of distress.

A stiffer foundation is achieved by using a spread-footing design that incorporates strips in a grid, as shown in Clemence and Finbarr (1981). In the place of interior isolated footing pads, additional strip footings are used. Longitudinal reaction beams span the spread footings to create a reinforced foundation grid that can withstand localized settlements from differential movements.

In both cases of continuous footing designs or grid-like foundations, the system can utilize either structural floors that are tied to the foundation elements or, if some movement is tolerable, floating slabs that are isolated from the foundation and load-bearing walls at specifically designed settlement joints. Because of the potential for movement with floating slabs, following specific drainage and irrigation recommendations is important to keep the subgrade soils as dry as possible.

An alternative to the extensive spread-footing grid system is a reinforced mat foundation, which consists of a monolithic concrete slab with continuous rebar reinforcement. The slab has reinforced ribs along the building perimeter and at locations of internal load-bearing walls or box-like spacing for rigidity and strength (**figure 9-9**). The reinforced mat foundation has the durability and strength to remain intact under most instances of differential settlement of the underlying soils. Although there could be a possible tilt toward the settled area, the settlements can be tolerated by the structure. The slab should be designed with sufficient strength to allow the slab to be releveled without damage, by using mudjacking or compaction grouting (Shaw and Johnpeer, 1985b).

Post-tensioned concrete slabs are commonly designed for multiple-unit developments in areas where the soil has a moderate collapse potential. Concrete is much stronger in compression than in tension, and post-tensioning is designed to take advantage of this. Post-tensioned slabs are constructed similarly to a concrete-mat foundation, but use high tensile-strength cables (tendons), rather than a rebar grid, to increase strength. After a certain cure period of the concrete, the tendons are pulled by a hydraulic ram, and locked at high-tension loads. This tension puts the concrete slab in compression so that it becomes much stronger and more rigid compared to a simple slab (**figure 9-10**).

An increasingly common technique that is applied in areas of both collapsible and expansive soils is the use of adjustable support columns or deck posts, which are tied to simple bearing pads that are not designed to counteract or mitigate ground movements. The adjustable support columns are used mostly for decks that are attached to buildings. Threaded bars (i.e., large bolts) are mounted to the bottom of deck posts and connected to a nut at the bearing pad that can be turned. The deck post can be simply releveled if ground movement occurs (**figure 9-11**).

Structural mitigation also includes reinforcing and double-sealing of "wet" utility lines (i.e., water and sewer pipes). The pipes may be reinforced to be more rigid or equipped with flexible couplings so they are better able to accommodate slight movements without rupturing and flooding collapse-prone soils. The location of ponds on collapse-prone soils is not recommended without properly designed, geotextile-reinforced, moisture-impermeable liners.

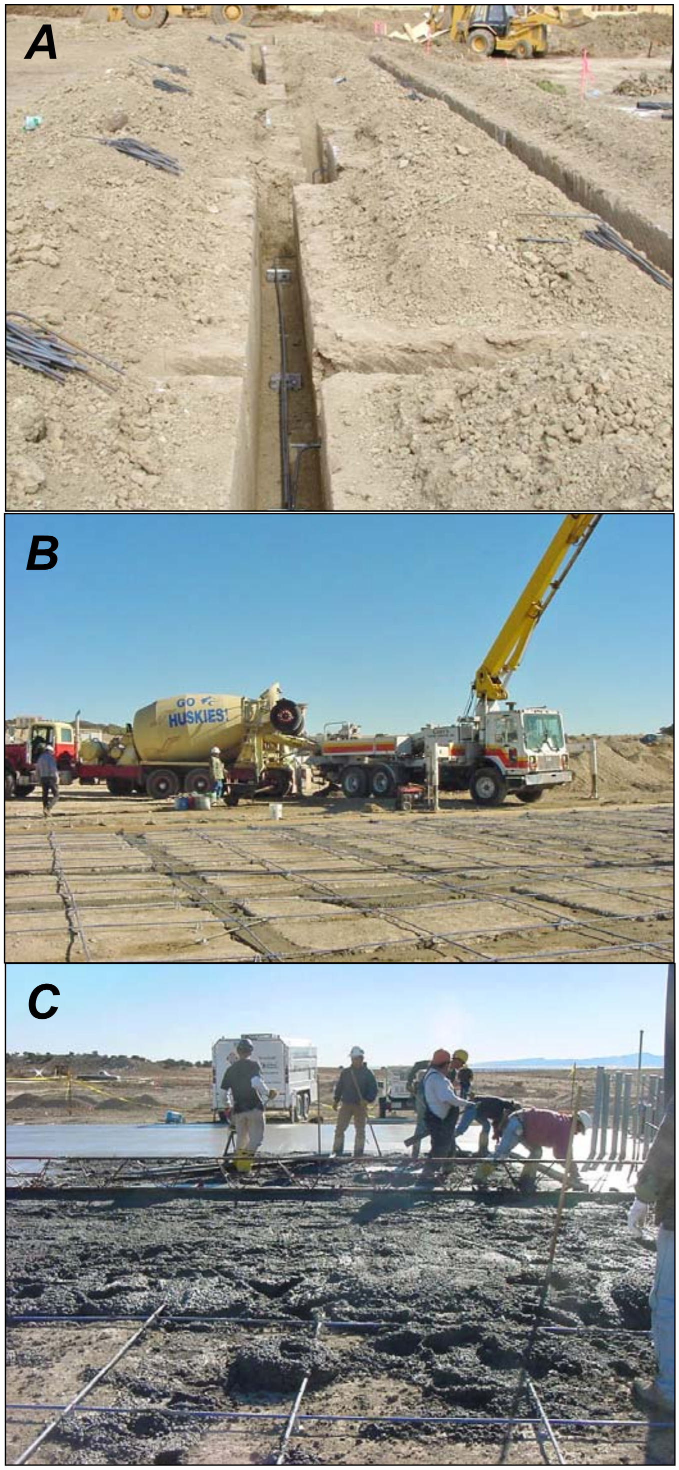


Figure 9-9. Reinforced concrete-mat foundation construction for a multiple unit structure. *A*, The prepared ground with excavations for the perimeter and internal ribs or grade beams of the mat. Note the steel reinforcement bars (rebars) at the bottom of the excavation. *B*, The placement of additional mat rebars over the box-like grid of the previous concrete pour. *C*, The final pour of the mat. Photos courtesy of Ron McOmber, CTL/Thompson, Inc.

Figure 9-10. Post-tensioned slab construction. *A*, The prepared ground prior to concrete pour. Note the grid of high-strength tendons protected in red sheathing. White spacers act as supports and ensure proper alignment of tendons within the concrete. *B*, After the concrete pour, the tendons that exit the side of the concrete slab are tensioned. The worker is operating a hydraulic pump that is forcing the hydraulic cylindrical ram near the worker's foot to pull the strand. Each tendon will be tensioned and locked at a designed force or load, thereby putting the concrete into compression. Photos courtesy of Ron McOmber, CTL/Thompson, Inc.

DEEP FOUNDATIONS

Deep foundations allow the transfer of the structural load through the soil horizon to a deeper stable substrate that has more favorable bearing capacity. A deep foundation may be the best solution when the potential of settlement from collapsible soils is very high, avoidance is not an option, the section of susceptible soil is thick, or the structure's tolerance for settlement is low. Deep foundations commonly are drilled shafts (caissons) and cast-in-place concrete piers or driven piles (emplaced with a pile driver or vibratory driver) that come in many configurations (e.g., H-piles, pipe piles, timber piles, tapered piles). Specialty types of deep foundations, such as helical-screw piers and hydraulically pushed or hammered micropiles, are generally used to underpin foundations in remedial repair, but can be used also for new construction.

There are special engineering considerations with the design of deep foundations in collapsible soils:

Figure 9-11. Adjustable deck post at townhome development in Basalt.

- With the use of deep foundations, later ground settlement could produce a gap under the pile caps and grade beams if floating-slab floors are used (Houston and Houston, 1989). Floating slabs could settle an unacceptable amount, as well. Grade beams and structural floors are generally recommended with deep foundations.

- The pile or shaft must be advanced through the entire column of collapsible soil. The geotechnical engineer must determine a worst-case scenario of potential wetting of the soil column and possible negative skin friction; the engineer must then specify the necessary foundation depth for the positive friction and (or) end-bearing capacities needed for the applied load of the structure. There are examples of adverse settlement of deep, friction-pile foundations in Colorado because of insufficient depth of placement. As the wetting front in the soil deepened, the soil collapsed, and settlement pulled the foundations with it.

REMEDIAL REPAIR MEASURES

Techniques for remedial repair of a settling foundation are limited because the structure is already in place and is already in various stages of distress and damage. Also, typical foundation walls generally do not have the strength to be releveled other than by a costly, incremental process. Instead, the damaged foundation is stabilized or, if the damage is too extensive and the amount of settlement is unacceptable, actually replaced.

Of the mitigation methods mentioned earlier, several are useful as remedial techniques. Compaction grouting is a common remedial technique and is quite effective in preventing further foundation movements. Mudjacking is effective for slabs-on-grade if the section of collapsible soils is thin. However, this treatment would likely need to be reapplied for thicker columns of soil because it further loads the soil and does not address the soil's continued collapse potential. Another applicable remedial method includes soil stabilization by injection of a sodium silicate solution (Rollins and Rogers, 1994). The rapid cementation of the collapsible soils could make this technique a valid remedial method if damage is determined early. Chemical stabilization would not require the extensive excavation or disturbance typical of other methods.

Foundation Underpinning

Underpinning is probably the most common form of remediation for settling foundations and internal slabs. In this process, narrow steel bars, cylindrical pins (pin-piles or micropiles), or helical piers are hammered, hydraulically pushed, or drilled into the subsoil to underpin and further support the structure. For foundations, the backfill is excavated to expose the foundation wall, and steel brackets (**figure 9-12**) are mounted at spacings determined by a structural engineer. The apparatus that advances the pile is then attached to the foundation wall (**figure 9-13**). The pipe is threaded to allow additional pipe segments to be added as needed. At the prescribed tip elevation of the pile (or at the point of practical refusal of further advancement), the pile is permanently attached to the foundation at the mounting bracket.

In designing underpins for mitigation of settlement related to collapsible soils, two important factors must be considered:

1. There should be an assurance that the pin-pile or micropile is advanced completely through the collapsible-soil column to a dense substrate with adequate bearing capacities. In reported instances where underpins were too short, either the wetting front in the soil eventually extended beyond the underpins or the negative skin friction exceeded the load capacity of the denser substrate, and settlement of the foundation resumed.
2. Underpinning methods may not be the best remedial choice in locations where there could be difficulty advancing the pile through rocky soils, such as hillside colluvium and gravelly layers in alluvial fans.

Controlled Wetting

Remedial controlled wetting, which induces collapse of the soil, is comparable to prewetting, except that it is undertaken after settlement of a foundation has occurred. This technique is usually an attempt to relevel a structure that has already experienced distress from differential settlement due to soil collapse. Water is introduced to that part of the subgrade that had not been wetted in an attempt to induce the same level of settlement. Extreme care is needed in controlling the location, amount, and rate of wetting. The quantity of water would need to be measured and added in increments as the structure is carefully surveyed and moisture contents of the soil are monitored. The controlled-wetting method has strong potential to aggravate problems because subsurface infiltration pathways are not predictable, so it is generally not recommended in residential neighborhood situations. It is not known whether this technique has been used in Colorado.

Figure 9-12. Excavation and attachment of steel bracket to a foundation wall using concrete expansion bolts. A steel pin pile is in the frame. Note settlement distress crack in the foundation wall. Photo courtesy of Steve Pawlak, HP Geotech, Inc.

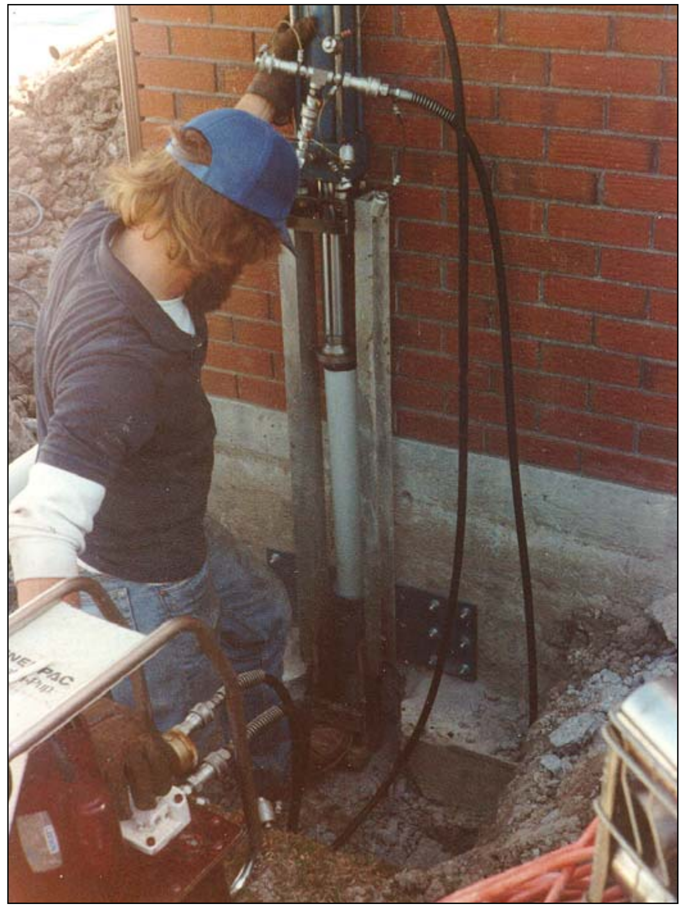


Figure 9-13. Piston apparatus, temporarily mounted on steel bracket, and white pipe pile being hydraulically advanced into the soil. Worker has his left hand on the hydraulic piston while operating the hydraulic pump, visible in the lower left corner of the photograph. Photo courtesy of Tom Griepentrog, Buckhorn Geotech.

Slab Demolition and Replacement

If a foundation is intact or if remedial work has been successfully completed, but the internal slab-on-grade floors were too badly damaged to be mudjacked or lifted by other means, it may be necessary to demolish and remove the concrete, prepare the subgrade, and repour the new slab. This can be costly and extremely disruptive, however. Pneumatic jackhammers would be required, which can generate extensive amounts of concrete dust that could settle throughout the structure.

Overbuilding New Floor on Existing Settled Slab

Another alternative if the existing slab is too badly distressed would be to build a new floor on top of it. Precise construction would be needed to level a new floor on an existing floor that has differentially settled (**figure 9-14**). Some assurance would be needed that there would be no additional movement in the future of the existing, damaged subfloor. In many circumstances, a new floor is constructed after underpinning, compaction grouting, or other remedial repair has been completed.

Foundation Demolition and Replacement

If the foundation settlement is so severe that damage to the foundation wall is extensive but the wood frame of a home is still intact, it may be economically feasible to temporarily lift the home and completely replace the foundation. The house can then be reset on the new foundation that is designed to mitigate collapsible-soil problems. A structural engineer would need to be retained to examine the feasibility of this alternative, as with any other structural modification.

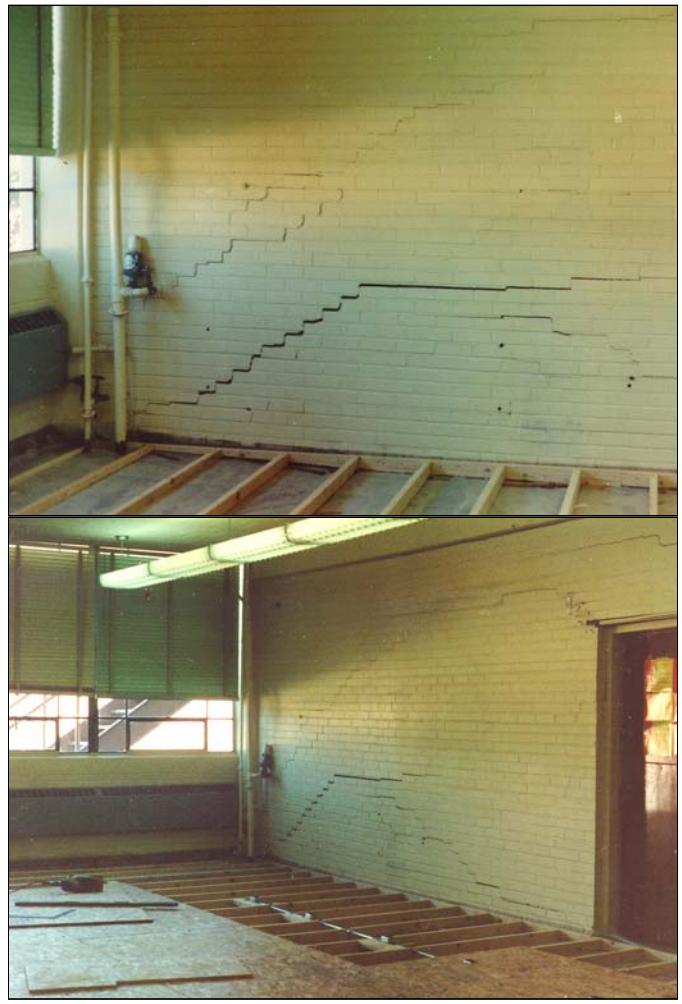


Figure 9-14. New floor being constructed on top of an older, distressed concrete slab at a school in Montrose. Note the distress in brickwork and the variable thickness of the new floor studs to accommodate settlement differentials. Photos courtesy of Tom Griepentrog, Buckhorn Geotech.

10. WHAT A HOMEOWNER OR PROPERTY OWNER CAN DO TO LESSEN THE RISK OF SOIL COLLAPSE

It is important to educate homeowners and property owners who might own homes or are considering buying homes or property sited in terrain where the soils could be susceptible to collapse, either by settlement of hydrocompactive soils or pseudokarst collapse by piping erosion or soil dispersion. Realtors and sellers are required by law to disclose preexisting problems to a buyer. However, these parties may be unaware of problem soils or may be unwilling to divulge details. It is incumbent on the buyer to research the geologic conditions of a site and learn whether mitigation has been implemented.

HOMEBUYER'S GEOTECHNICAL INSPECTION GUIDE

Table 10-1 is a suggested homebuyer's geotechnical inspection guide that was revised from Shelton and Prouty (1979) to be more specific to collapsible-soil susceptible areas in semiarid climates. Common misconceptions of property owners and prospective property owners concerning geotechnical evaluations were laid bare in table 1-1.

In considering raw real estate purchases, problems can be prudently avoided. If the property is located within a collapse-susceptible area, based on available maps, the prospective owner would be well advised to request or contract for a site-specific geotechnical evaluation prior to closing on a property. If collapse susceptibility is suspected, or the collapse potential of the soils is verified by testing, the buyer should have a clear understanding of the potential geologic hazards before purchase. The buyer should also try to determine whether the development plan accounts for the hazards that could be present. There is a common misconception that after approval of a project by the local planning and building departments, the geologic hazards have been addressed. That is not always the case. If collapsible soils exist on a property, a developer could seek approval under the conditions that 1) subject to approval, the soils would be designed for and mitigated on a lot-specific basis by the owner or builder or (2) wetting of the subsoils would be avoided (a condition that is difficult to manage for the lifetime of a residential structure).

Additional caveats are associated with new construction. After lots have been approved for sale, the initial concerns raised about the soils—or about any geologic hazard for that matter—could be ignored or forgotten by the developer and real estate agents, so as to not adversely impact property values. The stipulation of site-specific foundation investigations is a standard procedure in jurisdictions where building departments require a stamp of a Professional Engineer licensed in the State. However, many times the foundation or structural engineer has not read or been provided with the preliminary geotechnical report that was prepared as part of the land-use application, if such a report was required in the first place. The local building department generally only provides an administrative review and does not ensure that the concerns raised in the original geotechnical report are covered in the site-specific foundation design.

The construction manager also needs to be informed of the potential soil hazards that could exist. Often, problems occur when the geologic hazard and geotechnical reports are overlooked or when the recommendations are not followed. In some cases, the engineering consultants have not been allowed to follow through on their recommendations to (1) inspect the foundation excavation, (2) redesign if there is a change of conditions, or (3) review the grading and landscaping designs, which could greatly affect the soil conditions. Postconstruction grading and irrigation practices by untrained and unaware landscape designers can also undo careful engineering and construction by introducing moisture to the subsoils. Mitigation methods to address collapse-prone soils will always be less costly when designed for new construction compared to the costs of remedial repair for an existing structure.

The most likely scenario whereby owners of an existing home become aware of the potential hazards of collapsible soils is when they start to notice adverse settlement of some part of their home and distress to the structure. If adverse settlement is observed in an existing home, it is essential to investigate all possibilities of water introduction immediately so that the advancing wetting front in the soil will slow and eventually stop (the most optimistic scenario). It might be necessary to pressure test water lines and inspect sanitation lines with a down-pipe camera. Unfortunately, much of the potential damage has already occurred by this time, and the approach becomes one of damage control and remedial repair, if needed. For those homeowners who are aware of the potential for soil collapse

Table 10-1. Homebuyer's Geotechnical Guide.

PROBLEM OUTSIDE HOME	SIGNIFICANCE	ACTIONS
Onsite lot grading		
Lot slopes toward structures; fill or embankments block natural drainageways; water ponds next to foundation.	Roof runoff, precipitation, and adverse irrigation will flow toward foundation, add water to subsoils, and aggravate collapsible-soil problems.	Regrade lot and foundation backfill so that the grade slopes away from structures.
Landscaping and irrigation		
Vegetation planted close to structure and foundation; nearby ponds; irrigation sprinklers near or splashing against foundation.	Heavy irrigation may add water to subsoils and cause same problems as poor lot grading. Irrigation lines near foundations, walls, and flat work will eventually leak.	Remove vegetation; control and reduce irrigation near structures; remove irrigation lines near structures; use Xeriscape alternatives.
Sewage disposal system		
Leach field is located uphill of structure; system is backing up; effluent is surfacing at leach field.	Migration of wetting to foundation and slab subsoils will aggravate collapsible-soil problems.	Pump septic tank; replace or relocate leach field.
Evidence of leaking water valves		
Moist location near spigot; shallow depression; dripping; staining of foundation wall.	Wetting of foundation subsoils will aggravate collapsible-soil problems.	Replace water valve or remove.
Storm-water management		
No roof gutters; damaged or leaking roof gutters, downspouts, and extenders or splash guards.	Roof runoff may be splashing or dripping directly against foundation walls and adjacent slabs, infiltrating and wetting subsoil.	Install roof gutters; replace or repair leaking or damaged roof gutters; install downspout extenders or pipe flows from downspouts.
Adjacent land		
Steep gully or arroyo nearby (within 300 ft); ground fissures; anomalous ground depressions	Piping voids may exist near or under home; spontaneous ground openings may occur.	Maintain positive grades and prevent concentrated runoff. For new home construction, overexcavation and replacement with nondispersive fill and (or) geotextile reinforcement layers.
Location on land sloping to valley side or toward mouth of drainageway in hills above.	Location may be on hillside colluvium or alluvial fan.	Verify subsurface soil conditions for collapse potential.
Exterior flat work; slab-on-grades		
Hairline cracks; no offset	Minor settlement or shrinkage	No problem; observe over long term.
Cracks with offset	Major settlement or heave	Check drainage issues or water introduction.
Arcuate cracking and settlement of flat work into subsiding ground depression; major tilting of slabs; evidence of previous leveling of slab	Soil collapse likely occurring or has occurred.	Check for leaking wet utility lines; adverse drainage barriers ponding water; or concentrating runoff. Relevel or replace flat work.
Exterior foundation walls (interior as well)		
Vertical or near-vertical cracks; horizontal offset in concrete.	Foundation has differentially settled	Keep surface and subsurface water away; determine structural damage; assess whether settlement will continue; repair as needed.
Utility connections		
Bending or tilting in water, power, or gas lines where they enter building.	Settlement may be straining utility lines; rupture is a possibility.	Replace or realign utility lines to relieve strain.
Exterior walls		
Cracks in masonry; separation has stair-stepping up brickwork; skew noticeable in window or door frames	Foundation has differentially settled	Keep surface and subsurface water away; determine structural damage; repair as needed

Table 10-1. Homebuyer's Geotechnical Guide continued.

PROBLEM OUTSIDE HOME	SIGNIFICANCE	ACTIONS
Ground floor or basement floor		
Wet floors; compression cracks in center of floor; buckling of slab-on-grade floors; displacement of ground floor at joint with foundation wall.	Settlement of the subgrade soils affecting flat work.	Check for interior water leaks; verify integrity of underground water and sewer lines; check whether foundation has also settled or only interior slab-on-grade floors; structural or cosmetic repair as needed.
Windows and interior doors		
Skew noticeable in frame; window or door not closing or opening properly (sticks in frame); angled gaps (batwings).	Foundation may be settling. Settlement is binding the sliding or opening mechanism.	As above; determine whether interior walls are load-bearing, floating, or placed on interior slabs; verify foundation has settled; needed repair may be structural or cosmetic.
Interior walls, floors, and ceilings		
Cracks with offsets in drywall; obvious crack repair; displacement in tile work; interior walls pulled down from ceiling; perceptible tilt in floors; ductwork pulled away from furnace.	Differential settlement of the structure; damage to load-bearing walls.	As above; check foundation walls and flat work for displacement; needed repair may be cosmetic or structural if damage becomes extensive.
Drains		
Is the ground floor or basement floor sloped to any interior drains?	With no drains, or with inadequate grading of flat work toward drain, accidental interior flooding will infiltrate floor and foundation wall joints and seep into subsoils.	Establish drains at location of sewer pipe in home.

but have not yet seen damage, controlling the introduction of water to the subsoil that underlies foundations, slabs-on-grade, and pavements is the most important objective in preventing adverse settlement and damage. There are certain proactive approaches to construction, irrigation, landscaping, and drainage improvements that can be implemented, which are also applicable to new and proposed structures. These same procedures are also important in mitigation of swelling soils. The CGS *Guide to Swelling Soils for Colorado Homebuyers and Homeowners* (Noe and others, 2007) discusses the prevention and control of water to the bearing subsoil, and many of the recommendations in that publication are also directly applicable to collapsible soils.

BACKFILL, FILL COMPACTION, AND SITE GRADING

After a foundation and a stem wall have been poured, the excavation must be backfilled. This procedure should be done in a controlled manner with a rammer compactor (jumping jack) or vibratory trench compactor, with the final ground surface sloping away from the structure (fig. 9-1). With improper compaction, the fill could subside against the foundation wall over time. Such settlement could result in a general slope of the land back toward the building, which could cause rainfall and surface flows to pool against the foundation wall and ultimately seep into the bearing soils. Such circumstances must be avoided. If underdrains are considered for basement construction, the drain gravel pack should be placed on an impermeable liner that is attached to the footing wall in a waterproof manner to create an impermeable barrier to the bearing soils. If the fill adjacent to the home has settled, it should be further compacted, and additional fill should be placed to reestablish a positive grade away from the foundation wall. If an exterior concrete slab poured against a foundation wall has tilted toward the structure, it should be mudjacked to a positive tilt away from the structure, if possible. If not, it should be removed and replaced. There are cases where backfill settlement, sags in concrete flat work, and settlement cracks have developed adjacent to new foundation walls before rain gutters were installed. For many case histories of collapsible-soil damage compiled for this study, basic drainage and water-management errors caused adverse wetting of foundation-bearing soils, which resulted in damaging settlement.

Overlot grading should not block natural drainage swales or result in a structure sited in a topographic depression. Misdirection of surface flows can cause water to collect or pool near structures, pavements, or slabs. **Figure 10-1** illustrates the effects of slopes on drainage (Noe and others, 2007). Problems can occur for driveway alignments, roadways, and homes that are placed in natural drainage paths, or for wide facilities on slopes where drainage cannot easily circumvent them. Many of the roadways in western Colorado (e.g., figs. 7-11 and 7-17) that are undergoing distress from differential settlement from hydrocompaction are a result of poor drainage of roadside ditches and wetting of the subgrade soils.

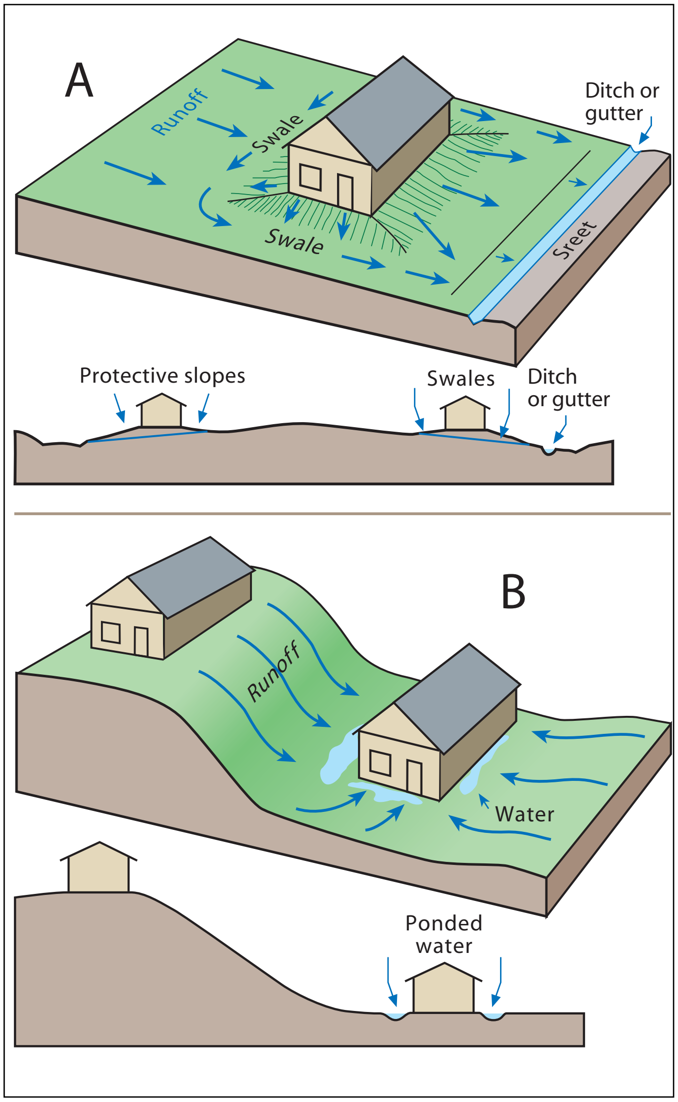


Figure 10-1. Effect of slopes on drainage. *A*, Carefully planned and maintained slopes provide positive drainage that prevents water from ponding near structures. *B*, Poorly planned slopes can result in poor drainage and ponding of water, which could result in soil saturation, collapse, and ground settlement. Image from Noe (2007).

IRRIGATION AND LANDSCAPING

In semiarid terrains of Colorado that may be susceptible to collapsible soils, irrigation should be limited. Irrigation and landscaping of residential, commercial, and public-facility lots almost inevitably results in an increase in moisture to the substrate, and care is needed in areas that are susceptible to collapsible soils. Excessive irrigation and leakage of irrigation water lines into the collapse-prone soils is one of the most common reasons for adverse settlement in Colorado. In a case history from the town of Meeker, a home was condemned in 1997 because a renter left a sprinkler on for several days near the house foundation, which deeply saturated the soils and dropped the foundation almost 6 in. in the center of that wetting zone (fig. 7-15). That this wetting also affected other homes down gradient indicates the importance of controlling both on-site and off-site water sources.

An alternative to irrigated lawns is a form of dryland landscaping called Xeriscape™, a term coined by Denver Water that means "water-wise landscaping," (from "Xeros," the Greek word for "dry"). It combines aspects of gravel cover, mulch, and planting of drought-resistant plants. There are many excellent resources to property owners who are looking for information on Xeriscape techniques; Noe and others (2007) provided an informative discussion that includes a list of water-wise plants. The official Xeriscape Web site is on the Denver Water Web site at http://www.denverwater.org/.

There are certain recommendations that should be followed with outside irrigation in areas prone to collapsible soils:

1. Irrigation water lines, valves, and mani folds should not be placed near foundations, concrete slabs, or pavements. A buffer of crushed rock or Xeriscape landscaping is suggested between the foundation wall and irrigated lawns or gardens.

2. Sprinkler heads should be installed at least 5 ft away from foundation walls, retaining walls, and slabs and pavements.

3. Sprinklers should not be allowed to spray against foundation walls, retaining walls, or pavement edges.

4. Irrigation water should not be allowed to pool, which could result in concentrated overland flow or infiltration into the under lying soil. This is a particular concern with dispersive or piping soils.

5. Timed systems are recommended to prevent deep saturation of soil from irrigation sprayers left on too long. An even better method is to use soil-moisture gauges to initiate a watering cycle. Moisture gauges have become cheaper after years of use in commercial and agricultural applications, and many of the new residential automated sprinkler systems may now be operated by these gauges. Their use significantly improves water management by replacing a blind timed system with one that recognizes whether a watering cycle is needed. It prevents the paradoxical and wasteful " watering in the middle of a rainstorm" that is commonly seen with timed systems.

6. Irrigation systems should be pressure tested periodically to ensure that leakage to subsoils is not occurring.

7. In terrain that is prone to soil collapse, water features such as ponds, ditches, waterfalls, etc. should not be located near structures or should be lined or sealed so that water is isolated from the subsoil. The impermeable liner or seal should be designed to accommodate some degree of movement without tearing or leaking.

The landscaping of existing homes may be retro-fitted with systems that meet these recommendations.

FACILITY AND SITE DRAINAGE

Proper facility drainage of storm-water flow is also very important, and grading and landscaping should be designed so that these flows drain the proximity of the home or structure as quickly as possible. However, it should be understood that the concentration of flows presents its own problems with erosion and dispersion (piping) in many parts of the State.

Roof gutters are necessary to avoid concentrating rainfall from the roof onto a narrow strip alongside a building. To put this in perspective, a ¼-in. rainfall on a 20-ft-long by 1-in.-wide roof segment with a 30° roof pitch would produce 52 cubic in. of water flowing from each 1-in. width of roof edge. That amount spread over a 2-ft-wide splash zone would be the equivalent of a 2.2-in. rainfall onto the ground immediately adjacent to the foundation wall, almost an order of magnitude higher than the original rainfall. **Figure 10-2** illustrates both proper positive backfill and use of rain gutters, downspouts, extenders, and splashblocks to control runoff and keep moisture away from structures (Noe and others, 2007).

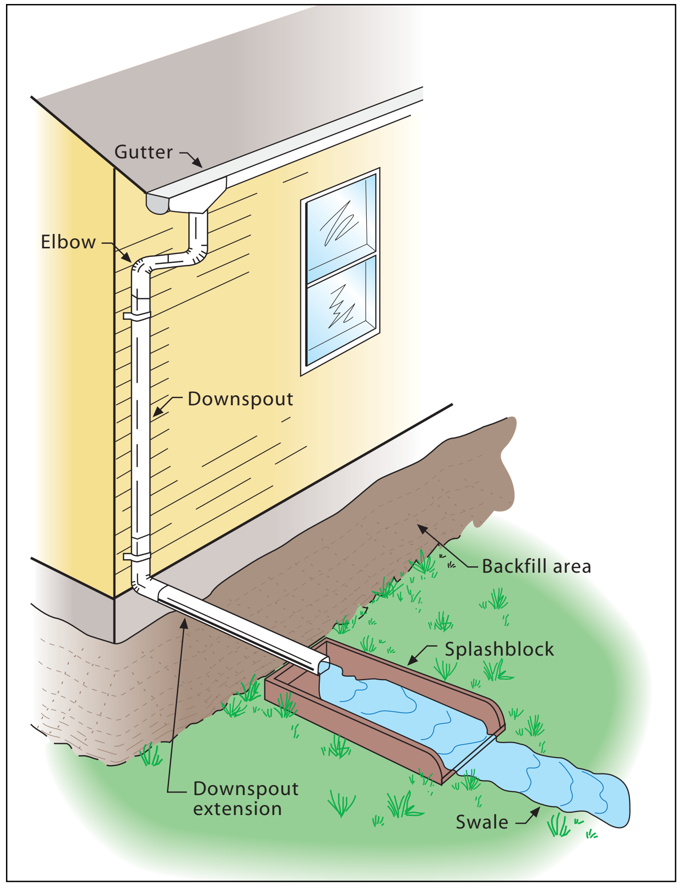


Figure 10-2. Recommended roof drainage (image from Noe, 2007). Note positive slope of backfill and use of downspout extender and splashblock to keep roof runoff away from structure foundation.

A Grand Junction garage illustrates the importance of rain gutters and adequate drainage (**figure 10-3**). Another example is the case of recently constructed townhomes in Glenwood Springs, a town well known for occurrences of collapse-prone soils (Morris and Weaver, 1978; Mock and Pawlak, 1983; Pawlak, 1998; White, 1998, 2002). The structures, built in 2001 to 2003, quickly experienced significant distress, which is partially attributed to lack of rain gutters and poor drainage on the back side of the structures (**figure 10-4**). Litigation soon followed and a $12 million settlement for the homeowners was reached in 2005 to repair the structures.

Rain gutters installed at the roofline will direct the entire volume of water that falls on an impervious roof to a downspout. Control of these downspout flows is very important. **Figure 10-5** illustrates the damage that may result when flows from a downspout saturate the ground near the foundation wall. Downspout extenders and positive grades are essential to convey rain flows beyond a point where they could migrate to foundation-bearing subsoils.

Figure 10-3. A large amount of localized settlement has offset the cinder blocks of this garage in Grand Junction. Note that there are no rain gutters for the roof. Photo by Ben Arndt.

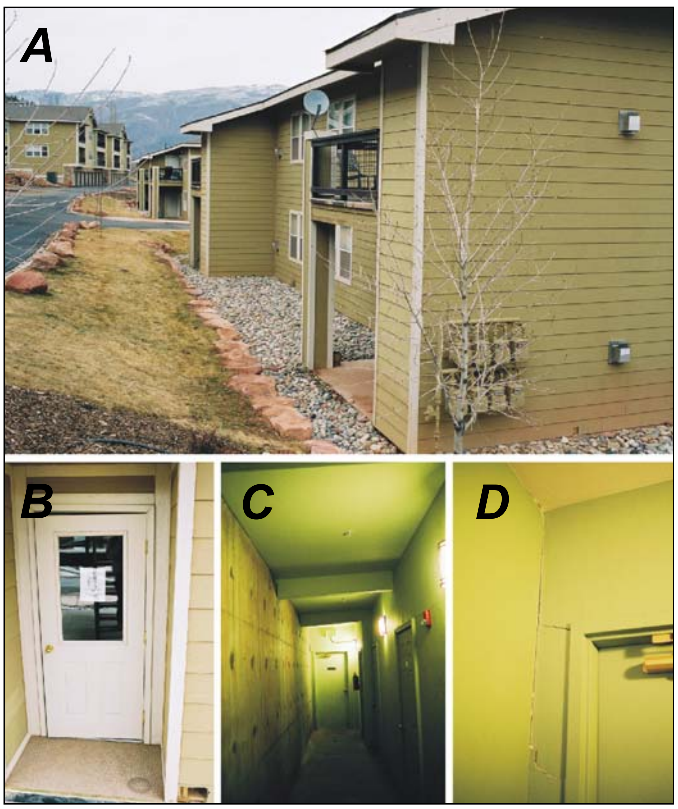


Figure 10-4. Distress of townhomes constructed from 2001 to 2003 in Glenwood Springs. *A*, Note the lack of rain gutters and the nearly flat and poorly drained, rocked-in area below the roof edges. *B*, Deflection of a doorframe. *C*, Deflected beams above a hallway as the concrete retaining wall on left is settling. *D*, Doorway with wall cracks forming as the wall is being pulled down in relationship to doorframe.

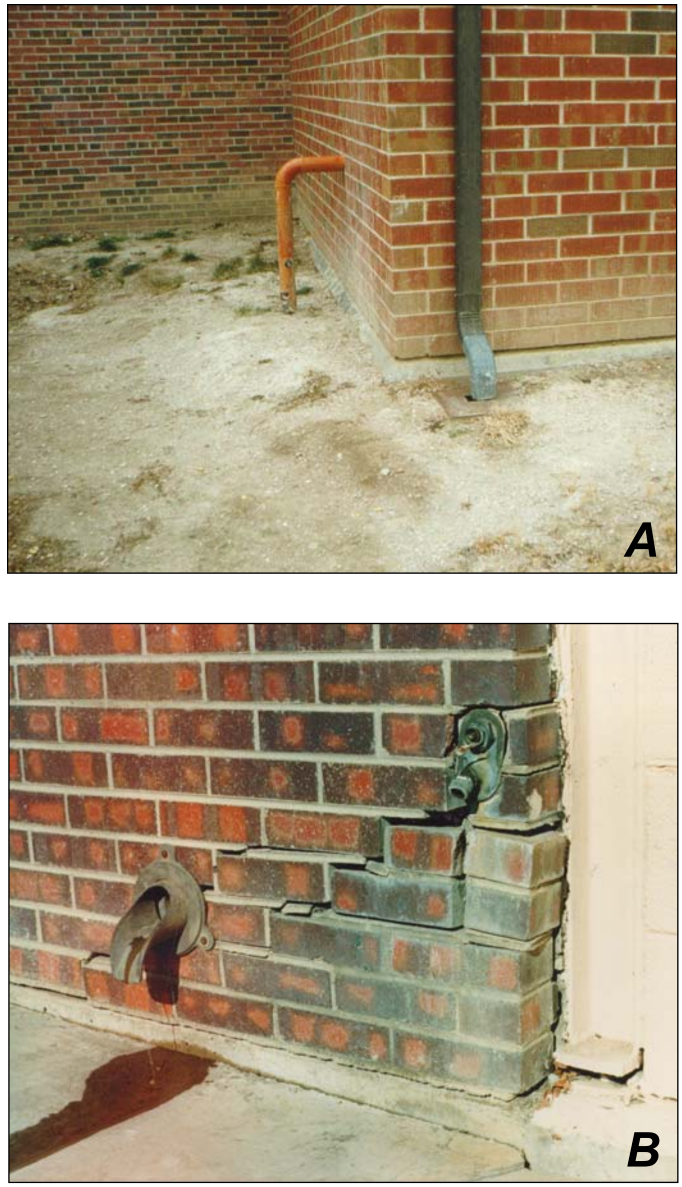


Figure 10-5. Damage to brickwork from ground settlement related to poor drainage. *A*, Note the gap between the brickwork and the concrete footing below at the location of the roof downspout at the Harriman School in Canon City. At the time this photograph was taken, the brickwork was still intact at this location. However, distress and signs of further foundation movement were seen in other parts of this school (fig. 3-7A). *B*, By comparison, this photograph shows a more severe effect of concentrated roof runoff that easily seeped to the subsoils. Note the offset of the sidewalk and foundation wall, which settled and resulted in severe distress of the brickwork. This structure at the Montrose Armory was condemned and demolished shortly after this photograph was taken. Photo courtesy of Tom Griepentrog, Buckhorn Geotech.

11. CONCLUSION

The preceding chapters have documented how collapsible soils can be destructive to foundations, roadways, bridges, dams, and irrigation structures when water is artificially introduced and adverse settlements occur. Homeowners, prospective property owners, and consultants working in certain parts of Colorado should be aware of the problems of collapsible soils, much as the public has gained an understanding of swelling soils along the Front Range. Not everyone is at risk. The property owner or geotechnical consultant can follow a checklist to determine the collapse potential of a site:

1. Does the property lie within the semiarid part of the State shown on plate 1, which is almost all of Colorado, except for mountainous terrain?

2. Is the site located in a geomorphic area characterized by either rapid deposition by water outside of a normal stream channel (for example, mud and debris flows on alluvial fans, sheetwash on hillsides, and alluvial overbank deposits) or deposition by wind?

3. Are the soils at the site composed of silt and clay, do the soils have a silt or clay matrix, or are they derived from fine-grained or evaporitic sedimentary deposits?

If the answer to these questions is yes, then a site investigation specifically oriented to collapsible soils should be performed, with the understanding that collapsible soils may be hydrocompactive, dispersive, or soluble.

Furthermore, from reading this report, the property owner or consultant should be able to dismiss those common misconceptions shown in table 1-1 of the Introduction, as well as make some rudimentary assessments by using the guide in table 10-1.

In a geotechnical investigation with a focus on collapsible soils, the types of soil tests performed depend on the future use of the site. At commercial sites with large structures, roadways, or sites to be used as dams or bridge abutments, the suite of tests may be different from those appropriate at residential sites. Even within the footprint of a residential building, the loading requirements for flat work are different from those for foundation loads, and soil properties can vary laterally. The limitations of various tests need to be understood in order to obtain the optimal information for design purposes.

Methods to address and mitigate collapsible soils include both ground modification and construction techniques. Remediation for damaged structures is possible and incorporates some of the same methods used in preconstruction mitigation. In all cases, proper water management—including the practice of maintaining effective surface and subsurface drainage—is critical to reducing the potentially destructive effects of soil collapse and ground settlement.

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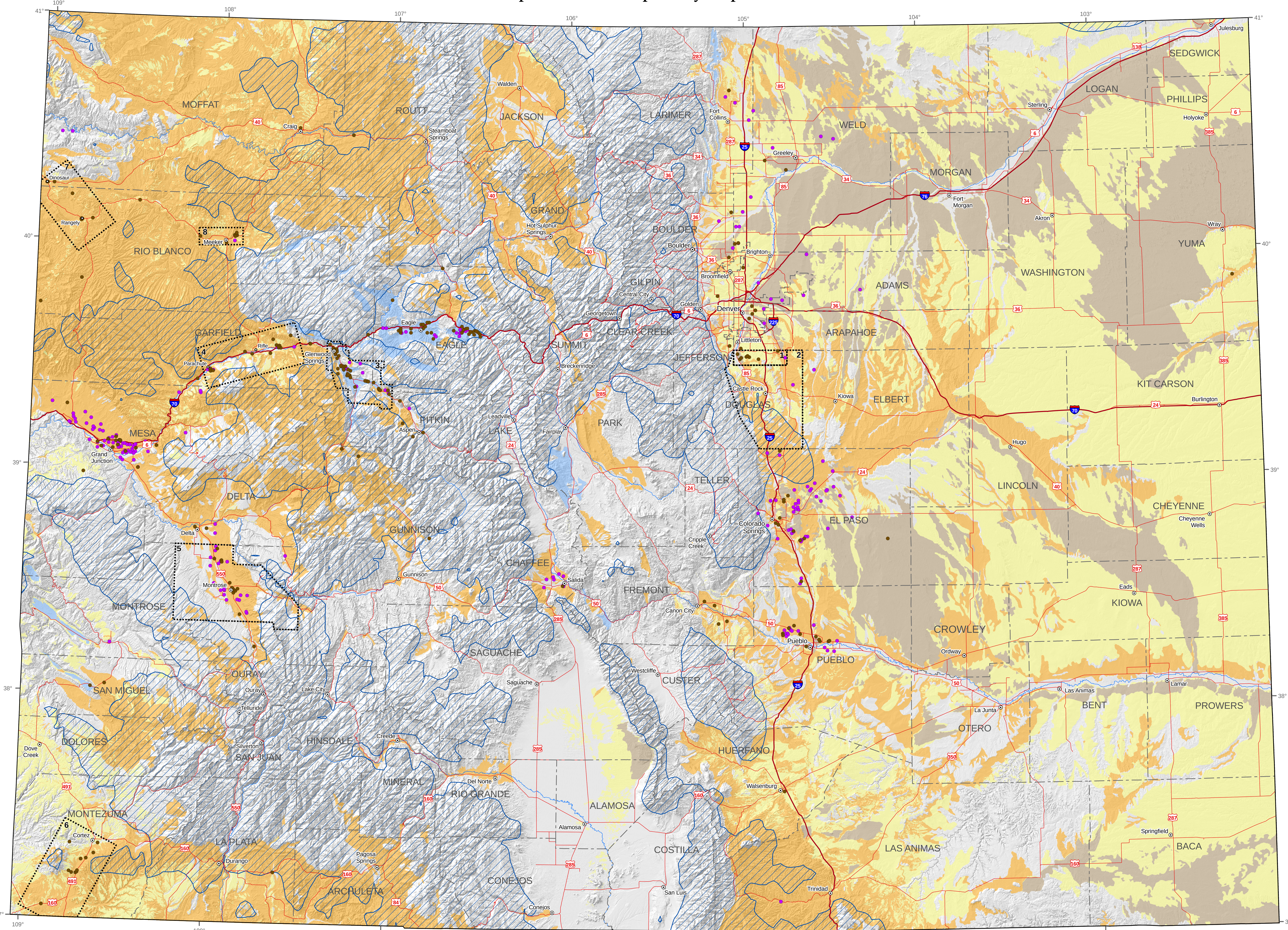
Collapsible Soil Susceptibility Map of Colorado



GIS and cartography by Nicholas Watterson

Scale = 1:1,000,000

25 0 25 50 Miles

25 0 25 50 Kilometers

Basemap data sources:
Colorado Department of Transportation
United States Geological Survey

Explanation

The purpose of this map is to show the general trends of where collapsible soils occur in Colorado. The map layers indicate those formational and unconsolidated units that are known to generate or contain sediments with soil-collapse potential. Unmapped areas outside the climate-exclusion zone may still contain areas where the geologic and geomorphic sediment-depositional conditions can produce collapse potential in unconsolidated soils. Site-specific investigations will best determine the collapse potential of a particular location.

Sedimentary Formations (Cretaceous and Tertiary)
These poorly indurated, bedrock formations with high percentages of clay and silt are easily eroded, and sediment yield is high. Both their composition and their softness lead them to be sources for sediments that may contain collapsible soils. In an arid to semiarid climate, unconsolidated Holocene (geologically recent) sediments derived from these formations can have the characteristics of collapsible soils when deposited in alluvial-fan, colluvial, alluvial, and eolian systems. Basic rock units are shown in the map plate stratigraphic column.

Eolian (Wind-Blown) Loess Deposits (Late Pleistocene and Holocene)
Eolian loess is composed primarily of clayey to sandy silt that accumulates as blanket deposits where the dust settles out of the air. This depositional process results in a soil structure with low density, high porosity, and a metastable open skeletal fabric. Loess soils are widespread on the eastern plains of Colorado but are less common in western Colorado, where they occur only in erosional remnants of what once were more extensive, ancient loess sheets that were deposited on top of flat or gently sloping mesa and pediment surfaces. Higher-elevation Pleistocene pediments and older terrace surfaces in western Colorado are typically mantled by loess sheets. Eroded loess sediments are also a source of collapsible soils when reworked and deposited in alluvial-fan, colluvial, and alluvial depositional systems. Certain loess soils of arid southwestern Colorado have very high percentages of secondary pedogenic gypsum, either dispersed or concentrated in specific soil horizons. Persistent wetting of such soils may cause dissolution of the gypsum and potential for long-term settlement.

Dune and Sheet-Sand Deposits (Late Pleistocene and Holocene)
Dune and sheet-sand deposits are generally not collapsible because the soil fabric is tight from the close packing of sorted sand grains. Collapsible characteristics can occur where the sand has appreciable windblown clay and silt (loess) concentrations, which can buttress the sand grains, prevent a tightly packed configuration, and create an open-fabric, metastable soil structure.

Evaporite Formations (Permian, Pennsylvanian, and Triassic)
Exposed evaporite rocks contain soluble minerals and are generally soft, poorly covered with vegetation, and prone to erosion. Highly collapsible soils are well known to occur in alluvial-fan and colluvial depositional systems where gypsiferous sediments have eroded from the evaporite formations. Evaporite bedrock was formed in the restricted basins of ancient shallow seas by the sequential evaporation and precipitation of minerals from supersaturated seawater. Soluble minerals such as halite, anhydrite, and gypsum were deposited, generally in association with fine-grained sediments of mud and nearshore sand.

Area with At Least 18 Inches of Annual Precipitation
The area of high precipitation is an exclusion zone for collapse-susceptible soils. In Colorado, at an approximate annual precipitation of more than 18 in., soil-saturation levels increase, and the potential of collapse comes to an end, even where the requisite clay- and silt-rich sediment sources and depositional systems exist. In general, the areas with these conditions are the mountainous terrains of the state. Though rare, collapsible-soil events have been reported within this area of 18-in. or more annual precipitation, generally on drier south to southwest slopes having heavy sun exposure and in alluvial fans. Spatial data from U.S. Department of Agriculture, Service Center Agencies.

Collapsible Soil Case History
Approximate location of collapsible soil occurrence compiled by the Colorado Geological Survey (CGS)

Collapsible Soil Location
Location of collapsible soil geologic hazard as noted in the CGS Land Use Review Database (2001-2008)

1 **Study Areas for Regional Collapsible Soils References**

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3. White, J.L., 2002, Collapsible soils and evaporite karst hazards map of the Roaring Fork Corridor, Garfield, Eagle, and Pitkin Counties, Colorado: Colorado Geological Survey Map Series 34, scale 1:50,000.

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7. White, J.L., in prep., Collapsible soil susceptibility map in the Rangely-Dinosaur vicinity, Rio Blanco and Moffat Counties, Colorado: Colorado Geological Survey Map Series, scale 1:50,000.

8. White, J.L., in prep., Collapsible soil susceptibility map of the Meeker vicinity, Rio Blanco County, Colorado: Colorado Geological Survey Map Series, scale 1:24,000.

Era	Period		Western Colorado	Physical Description	Front Range	Physical Description
Cenozoic	Quaternary	Holocene	Recent surficial soil deposits	Unconsolidated sediments range from silty clay to sandy gravel depending on depositional environment	Recent surficial soil deposits, includes some eolian deposits and Piney Creek Alluvium	Unconsolidated sediments range from silty clay to sandy gravel depending on depositional environment, some loess
		Pleistocene	Most eolian deposits and unconsolidated deposits of the Pindale and Bull Lake glacial epochs (includes Redlands Alluvium near GJ)	Mesas, terraces, and pediments capped with river-deposited gravel and sand	Unconsolidated glacial epoch gravel and sand alluviums: Broadway Alluvium, Louviers Alluvium, Slocum Alluvium, Verdos Alluvium, Rocky Flats Alluvium, Nussbaum Alluvium	Terraces, mesas, and pediments capped with river-deposited gravel and sand
				windblown loess	Mixed eolian deposits on the plains	dune sand and windblown loess on the plains
	Tertiary		Browns Park Formation	Sandstone and siltstone	Dry Union Formation	Siltstone and sandstone
			Uinta Formation	Sandstone and siltstone	Castle Rock Conglomerate	Conglomerate and sandstone
			Green River Formation	Shale, oil shale, siltstone, and marlston		
			Wasatch Formation	Shale, siltstone, and sandstone	Dawson Formation	Sandstone and interbedded siltstone and shale
Mesozoic	Cretaceous		Mesa Verde Group	Sandstone and interbedded siltstone, shale, and coal	Denver Formation, Arapahoe Formation, Laramie Formation, Fox Hills Sandstone	Sandstone and interbedded siltstone, shale, and coal
			Mancos Shale, Frontier Sandstone Member	Shale, siltstone, and minor sandstone	Pierre Shale	Shale
					Niobrara Formation, Colorado/Benton Group	Limestone and limy shale, Shale
			Dakota/Burro Canyon Formation	Sandstone and interbedded shale	Dakota Sandstone	Sandstone
	Jurassic		Morrison Formation	Variegated mudstone and Sandstone	Morrison Formation	Mudstone, sandstone, limestone, and gypsum
			Entrada Sandstone	Sandstone		
	Triassic		Chinle Formation	Red shale and siltstone		
Paleozoic	Pennsylvanian/Permian		Southwest / West-central		Lykins	Red siltstone, limestone, gypsum
			Cutler Fm / Maroon Fm	Red sandstone, siltstone, and shale	Lyons, Fountain Formation	Sandstone, Red sandstone and interbedded mudstone and shale
			Hermosa Gp, Paradox Fm / Eagle Valley Fm Evaporite	Siltstone, shale, thick gypsum and salt beds		
			/ Minturn Fm	Sandstone, limestone, and shale with evaporite facies		

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