SPECIAL PUBLICATION 22

PROCEEDINGS of the 33rd ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

ENGINEERING GEOLOGY and ENVIRONMENTAL CONSTRAINTS

IN
VAIL, COLORADO
1982

JEFFREY L. HYNES, EDITOR

Colorado Geological Survey
Department of Natural Resources
Denver, Colorado
1983

SPECIAL PUBLICATION NO. 22

PROCEEDINGS OF THE 33RD ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

Engineering Geology and Environmental Constraints in Vail, Colorado 1982

Jeffrey L. Hynes
Editor

Colorado Geological Survey
Department of Natural Resources
State of Colorado
Denver, Colorado

1983

$9.00

33rd ANNUAL HIGHWAY GEOLOGY SYMPOSIUM
SEPTEMBER 15-17, 1982
VAIL, COLORADO

ORGANIZING COMMITTEE

Martin C. Everitt, Chairman
Regional Geotechnical and Materials Engineer
USDA Forest Service
Rocky Mountain Region

Doyt Y. Bolling
Director, Office of Construction and Maintenance
Region 8, Federal Highway Administration

Brandon Gilmore
Chief Engineering Geologist
Colorado Department of Highways

Jeffrey L. Hynes
Senior Engineering Geologist
Colorado Geological Survey

Donald D. Loetterle
Engineering Geologist
USDA Forest Service
Rocky Mountain Region

Richard J. Suedkamp
Project Manager
Chen and Associates, Consulting Engineers

The sponsorship of the 33rd Annual Highway Geology Symposium was unusual in that no single organization undertook complete sponsorship of the meeting. The above named individuals undertook the organization informally. Special thanks are due to their employers for making available the time and support services required.

BANQUET SPEAKER

Alan F. Huggins
District Vice President
International Engineering Company
"The Road We Couldn't Build"

HIGHWAY GEOLOGY SYMPOSIUM:
Its History, Organization, and Function

Born of the need to establish a better understanding and closer cooperation between geologists and civil engineers in the highway industry, the Highway Geology Symposium was organized and held its first meeting on February 16, 1950 in Richmond, Virginia. Since then, 31 consecutive annual meetings have been held in 22 different states. Between 1950 and 1962 the meetings were held east of the Mississippi River, with Virginia, Ohio, West Virginia, Maryland, North Carolina, Pennsylvania, Georgia, Florida, and Tennessee serving as the host states.

In 1962, the Symposium moved west for the first time to Phoenix, Arizona. Since then, it has rotated, for the msot part, back and forth from east to west. Following meetings in Texas and Missouri in 1963 and 1964, the Symposium moved to Lexington, Kentucky in 1965, Ames, Iowa in 1966, Lafayette, Indiana in 1967, back to West Virginia at Morgantown in 1968, and then to Urbana, Illinois in 1969. Lawrence, Kansas was the site of the 1970 meeting, Norman, Oklahoma in 1971, and Old Point Comfort, Virginia the site in 1972.

The Wyoming Highway Department hosted the 1973 meeting in Sheridan. From there it moved to Raleigh, North Carolina in 1974, back to the west to Coeur d'Alene, Idaho in 1975, Orlando, Florida in 1976, Rapid City, South Dakota in 1977, and then back to Maryland in 1978--this time at Annapolis. Portland, Oregon was the site of the 1979 meeting, Austin, Texas in 1980, and Gatlinburg, Tennessee in 1981. The 1982 meeting was held in Vail, Colorado in September. Meetings in 1983 and 1984 are planned for Georgia and California.

Unlike most groups and organizations that meet on a regular basis, the Highway Geology Symposium has no central headquarters, no annual dues, and no formal membership requirements. The governing body of the Symposium is a steering committee made up of about 20 engineering geologists and geotechnical engineers from state and federal agencies, colleges and universities, as well as private service companies and consulting firms throughout the country. Steering committee members are elected for three-year terms, with their elections and re-elections being determined principally by their interests and participation in and contributions to the symposium. The officers include a chairman, vice chairman, secretary, and treasurer, all of whom are elected for a two-year term. They may succeed themselves for one additional term.

A number of three-member standing committees handle the internal and external affairs of the organization. Some of these are: the By-Laws Committee, Public Relations Committee, Award Selection Committee, Publications Committee, etc. Committees are held to a minimum for the most part, however, to avoid bureaucratization of the organization. The lack of rigid specialization, requirements, and routine and the relatively relaxed overall functioning of the organization is what attracts many of the participants.

Meeting sites are chosen two to four years in advance and are selected by the Steering Committee following presentations made by representatives of potential host states. These presentations are usually made at the Steering Committee meeting, which is held during the Annual Symposium. Upon selection the state representative becomes the state chairman and a member pro tem of the Steering Committee. Depending on interest and degree of participation, the temporary member may gain full membership to the Steering Committee.

The symposia are generally set up for two and one-half days, with a day-and-a-half for technical papers and a full day for the field trip that usually occurs on the second day. In most cases the activities begin on Wednesday morning with the opening session. The field trip is ususally set for Thursday, followed by the annual banquet that night. The final technical session usually ends by noon on Friday.

The field trip is the highlight of the meeting. In most cases, the trips traverse from 150 to 200 miles, provide for six to eight scheduled stops, and require about eight hours. Occasional cultural stops are scheduled around geological and geotechnical points of interest. In Wyoming, for example, the group viewed landslides in the Big Horn Mountains; Florida's trip included a tour of Cape Canaveral and the NASA space installation; the Idaho and South Dakota trips dealt principally with mining activities; North Carolina provided stops at a quarry site, a dam construction site, and a nuclear generating site; in Maryland the group visited the Chesapeake Bay hydraulic model and the Goddard Space Center; the Oregon trip included visits to the Columbia River Gorge and Mount Hood; the Central Mineral Region was visited in Texas; and Tennessee provided stops at several repaired landslides in Appalachia.

At the technical sessions, case histories and state-of-the-art papers are the norm. Highly theoretical papers are the exception.

For more information regarding the Symposium, contact any of the steering committee members listed in this publication.

*HIGHWAY GEOLOGY SYMPOSIUM

Medallion Winners

Hugh Chase	- 1970
Tom Parrott	- 1970
Paul Price	- 1970
K. B. Woods	- 1971
R. J. Edmonson	- 1972
C. S. Mullin	- 1974
A. C. Dodson	- 1975
Burrell Whitlow	- 1978
Bill Sherman	- 1980
Virgil Burgat	- 1981
David L. Royster	- 1982
Henry Mathis	- 1982

*In 1969, the Symposium instituted an awards program, and with the support of Mobile Drilling Company of Indianapolis, Indiana designed a plaque to be presented periodically to individuals who have made significant contributions to the HGS over a period of years. The award, a 3 1/2" medallion mounted on a walnut shield and appropriately inscribed, is presented during the banquet at the Annual Symposium.

STEERING COMMITTEE MEMBERS
1982

	TERM EXPIRES
Dr. Terry R. West - Chairman Assoc. Professor Department of Geosciences and Civil Engineering Purdue University West Lafayette, Indiana 47907 Phone - (317) 494-8171	1982
Burrell S. Whitlow - Vice Chairman President Geotechnics, Inc. 231 Walnut Avenue Vinton, Virginia 34179 Phone - (703) 344-4569	1983
David L. Royster - Secretary Director of Soils and Geotechnical Engineering Tennessee Department of Transportation 2200 Charlotte Avenue Nashville, Tennessee 37203 Phone - (615) 641-4775	1982
W. D. Bingham - Treasurer State Highway Geologist Department of Transportation Division of Highways Raleigh, North Carolina 27611 Phone - (919) 733-6911	1984
William F. Sherman Chief Geologist Wyoming Highway Department P.O. Box 1708 Cheyenne, Wyoming 28001 Phone - (307) 777-7456	1983
Joseph A. Gutierrez Geologist Mideast Division Vulcan Materials Company P.O. Box 4195 Winston-Salem, North Carolina 27105 Phone - (919) 767-4600	1982
Henry Mathis Assistant Director Division of Materials Geotechnical Session Kentucky Department of Transportation Frankfort, Kentucky 40601 Phone - (502) 564-3160	1983

R. G. Charboneau 1982
Chief Geologist
Idaho Department of Highways
P.O. Box 7129
Boise, Idaho 83707

Ed J. Zeigler, Associate 1983
Rummel, Slepper, and Kahl
1035 N. Calvert Street
Baltimore, Maryland 21202
Phone - (301) 247-2260

Vernon L. Bump 1984
Foundation Engineer
Department of Transportation
Division of Highways
Pierre, South Dakota 57501
Phone - (605) 773-3401

David Mitchell 1984
Soils Engineer
State Highway Department of Georgia
Forest Park, Georgia 30050
Phone - (404) 363-7520

L. Edwin Garner 1982
Geologist
Bureau of Economic Geology
University of Texas at Austin
University Station, Box X
Austin, Texas 78712
Phone - (512) 471-1534

Berke Thompson 1982
Assistant Director
Materials Control, Soil and Testing Division
West Virginia Department of Highways
312 Michigan Avenue
Charleston, West Virginia 25311

Dr. Michael A. Ozol 1983
Research Institute for Advanced Studies
1450 S. Rolling Road
Baltimore, Maryland 21227
Phone - (301) 247-0700

Mitchell D. Smith 1983
Engineer
Research and Development Division
200 N.E. 21st Street
Oklahoma City, Oklahoma 73105
Phone - (405) 521-2671

Gerald E. Wallace 1983
District Geologist
Missouri State Highway Department
Town and Country, Missouri 63131
Phone - (314) 966-3800

W. A. Wisner 1984
Geologist
Florida Department of Transportation
Office of Materials and Research
P.O. Box 1029
Gainesville, Florida 32601
Phone - (904) 372-5304

Walter F. Fredericksen 1982
Kansas Department of Transportation
R.R. #2, Box 13B
Erie, Kansas 66733
Phone - (316) 244-5530

John B. Gilmore 1983
Colorado Highway Department
4340 East Louisiana
Denver, Colorado 80222

George Meadors 1983
Virginia Department of Highways and Transportation
1221 East Broad Street
Richmond, Virginia 23219
Phone - (804) 786-4715

Harry Ludowise 1983
Federal Highway Administration
610 East Fifth Street
Vancouver, Washington 98661

Newly elected to the Steering Committee:

Jeff Hynes - Colorado Geological Survey

Willard McCasland - Materials Division, Oklahoma Department of Transportation

Marvin McCauley - California Division of Highways

TABLE OF CONTENTS

Welcome and Opening Remarks

Highway Geology in Colorado And Overview

Abstract

by

J. W. Rold
Colorado Geological Survey
Denver, Colorado

Geologic factors are a major control mechanism for all aspects of Colorado highways. Both the regional and the local topography in Colorado directly result from geological controls. This control ranges from our major valleys and mountain ranges to the smallest canyons. Because of the scale of these major topographic units in Colorado, the major transportation routes themselves--from the days of the Indians and early explorers, to today's interstates--are directly controlled by the regional and local geology. Since early historic times, topography, the soils, and the mineral resources have controlled the location of the population and the service centers, and therefore, the requirements for a transportation system.

Colorado's mineral resource locations, which are obviously governed by the geology, have had a definite shape on our transportation needs. Had gold been discovered at the confluence of St. Vrain and the South Platte instead of Cherry Creek and the South Platte and early gold production established in the St. Vrain or Poudre drainages instead of Clear Creek, the major metropolitan area and capitol of Colorado might well have been Platteville instead of Denver.

The location, character, and intensity of major canyon routes and mountain passes is definitely controlled by the tectonics, the rock character, the geological history, and erosional processes. Not only does the regional geology control the general location of our transportation, and therefore highway corridors, local geologic factors and processes control the specific

location, designed feasibility, and the cost of not only construction, but also the maintenance and the safety of the roadway.

These factors could be classified under rock characteristics, processes, and hydrology. Specific rock characteristics are hardness or strength, which determine the stability, height, slope, and shape of cut slopes, erodability, bearing strength, rippability, structure relates to dip, strike, and bedding of sedimentary or layered units, foliation, faults, and joints. Stability relates to shrink swell, hydrocompaction, and mine subsidence. Geologic processes include landslides, mud and debris flows, rockfall, avalanches, seismicity, and erosion. Hydrology relates to flood plains, water saturation, seasonal changes, drainage, and hydrology's impact on stability, particularly in landslides and swelling soils.

The manner of investigating, predicting, and mitigating the effects of these geologic factors controls the economics of construction and maintenance and, more importantly, the safety of our highways.

This symposium should address many of these ubiquitous and unique problems of Colorado highways.

Experimental Compaction of Collapsible Soils at Algodones, New Mexico

by

Warren Bennett

Geotechnical Engineer

State of New Mexico Highway Department

In order to determine the efficacy and capabilities of various potential methods of compacting collapsible (hydro-compacting) soils, four methods of compaction were tried in a test program under controlled conditions. The test series included both minimal efforts and established commercial techniques never used on these soils. They included forced wetting, without and with vertical drain channels, and the use of vibroflotation and impact compaction. In addition to determining which methods were effective, the experiment was intended to make the potential contractors and subcontractors aware of the capabilities and limitations of their methods to refine the bidding and maximize the returns.

In addition to explaining the trial methods and the results obtained, the results of the bidding and construction procedures on the two production projects will be discussed, including the methods selected, the specifications used and the apparent savings.

INTRODUCTION

Many of New Mexico's highways have been plagued by soil collapse since rigid and semi-rigid or flexible pavement have been used for roadway surfaces. In many areas, thousands of dollars in maintenance monies are spent each year to re-level roadway surfaces after millions of dollars have been spent for primary construction. Some of these roads were built before the phenomenon of soil collapse was recognized. The primary concern in areas suspected of having collapsible soils is in identifying the problem. A secondary concern is to use a method of soil stabilization that will be economically feasible, and prevent future expensive maintenance costs throughout the lifetime of the roadway.

The recognition of the phenomenon of soil collapse or hydrocompaction, is relatively recent, and the amount of available literature for identifying, testing, and treating, although growing rapidly, is limited. These type soils occur in many parts of the world, including the USSR, Bulgaria, Israel, Czechoslovakia, Argentina, South Africa, Hungary, Rumania, and the United States. The classic type section, at least in the United States, probably is in the San Joaquin Valley in California, where collapse in excess of ten feet occurred during pre-construction research for the San Luis Canal. The phenomenon has also been observed in the remaining southwestern states as well as in loessial deposits in Iowa and Nebraska. Other areas in the United States may also have such soil conditions.

Collapsing soils are those that undergo an appreciable loss of volume upon wetting, load application, or a combined effect of both. In the arid southwest they are primarily fine-grained silts, and sands with minor amounts of colloidal materials or dissolved salts, which act as bonding agents. They may be transported and deposited by the wind as loessial deposits or dune silts and left in a dry, loose state in arid regions where there is insufficient moisture for hydrocompaction. They also may be transported by sheet wash or mud flows and deposited on low-gradient land forms such as a bajada, alluvial apron or river floodplain, where the velocity of sheet flow diminishes enough for soil deposition, yet the water and most of the finer grained particles continue to flow to lower parts of the environment. The resulting formation is a low density soil weakly bonded by a small percentage of colloidal particles or salts which dry, form a weak crust, and prevent deep penetration of water for natural compaction.

Typically, the arid weather cycle consists of intermittent flash floods, when a few inches of soil are deposited, followed by extensive dry periods, when practically all of the soil moisture evaporates from the older deposits as well as the latest accumulation. Consequently, no soil layer has to support more than a few inches of load while in a wet state. Over the ages, as the landscape is exposed to hundreds, perhaps thousands, of these cyclical events, many feet of low density materials accumulate.

When the landscape is changed, either naturally or artificially, excessive moisture may infiltrate these low-density soils, dissolve or weaken the clay or salt bonding properties, and cause the soil to collapse from its own weight. Saturation simply allows the soil particles to rearrange themselves to a higher density state. Also, the soil may collapse from the additional weight of a man-made structure such as a roadway fill, bridge, or building, simply because the applied stress intensity may break the cementing bonds of the colloidal materials.

One of the first problems regarding collapsible soils in New Mexico was noticed in the late sixties at the maintenance patrol yard at Alcalde, between Espanola and Velarde. Several inches of differential settlement were noticed at the west end of the shop building after planting and watering a lawn, and in the southwest corner of the building after maintenance personnel began washing equipment there. The patrol foreman reported that he also had similar extensive damage to his personal dwelling, an adobe house built on similar soils, after landscaping and watering his yard. Other sites of known distress related to soil collapse are: portions of NM 68 between Alcalde, and Velarde, which have sunk up to three feet in relatively long arcuate sags, and a bridge with a slab footing on NM 22 over the Santa Fe Railroad near Santo Domingo which developed a badly cracked pier cap. Continual track subsidence occurred in the railroad when water was directed from a paved approach fill to the vicinity of the pier. The footing of the damaged pier is founded on collapsible soil in a stream meander. More sites include: the SHD District 3 office buildings in Albuquerque which were severly damaged from soil collapse following landscape watering; numerous public and private buildings in and around Albuquerque, and NM 422 and US 85 (now designated I-25) near Bernalillo and Algodones - the latter being the subject of this paper.

THE PROJECT SITE

In 1979, a zone of "collapsible" soils about 3 1/2 miles long was identified on Interstate 25 at Algodones, N.M., about 20 miles north of Albuquerque in the Rio Grande valley. This portion of I-25, although designated a part of the Interstate System, had not been brought to Interstate Standards. The primary construction plan included frontage roads to be let to contract as the first part of a phase construction program, and mainline construction to be let to project for stabilizing the collapsible soils on the frontage road would be quite feasible, and that it would provide valuable information as to the most economical method for stabilization of the soils on the mainline construction.

A request was made for the Federal Highway Administration to participate in this experiment, and it was undertaken as a joint venture in late 1979.

The terrain from Bernalillo to the junction of NM 422 and old US 85, rises and falls from alluvial apron or coalesced fan deposits to terrace deposits back to apron deposits. The terrace deposits are an erosional feature, a result of entrenchment of the Rio Grande and its tributaries, and the apron deposits are primarily a constructional feature built as described under the sheet-wash hypothesis where collapsible soils accumulate.

The principle area of concern, i.e. the area showing the greatest amount of distress, lies in the last three and one-half miles of apron deposits, beginning at the toe of the terrace deposit south of the junction with NM 474 and ending a few hundred yards north of the Junction of NM 422 and old US 85. The site chosen for the experiment lies about the middle of this section of apron deposits on the frontage road construction, opposite the Plains Electric Power Plant.

The apron deposits are not a homogeneous mass of collapsible soil. They have been breached by intermittent stream braiding and contain strands of hydrocompacted materials which are usually much more coarse-grained than the collapsible soils. The collapsible zone of the profile varies from 20 to 25 feet in thickness. The primary soil types in the profile are nonplastic silt (A-4) and silty sand (A-2-4), with minor stringers of sandy gravel (A-1-b).

New Mexico 422, the precursor of Interstate 25 in the Algodones area, was built in 1955. Records of the first several years of service are scant, but according to the memory of those polled, the undulating, wavy surface began to develop in the pavement within a year or two after construction. Sags and wavyness have continued to form throughout the life of the roadway, and periodic scab patching and leveling of the wavy surface have been required in order to maintain a decent ride. Collapse of the interfluvial low density soils accounts for the wavy configuration in the roadway surface.

In the late sixties, during the routine preliminary soils sampling and testing phase of the profile investigation, it was recognized that a soils problem existed at Algodones, but no one was quite sure of what it was. Several designs were run on shallow seated soils using lime and cement as a stabilizing agent. Neither of these two products were satisfactory for solving the problem.

Because of problems in land acquisition the preliminary engineering soils work for this part of I-25 was delayed and no further final materials design work was done until the latter part of the seventies. In late 1978 the problem of soil collapse was positively identified.

A drilling program was undertaken through the problem area, numerous holes were drilled adjacent to the sags and waves in the existing road and in undisturbed soils along the frontage road alignment. Standard penetration, soils classification, and moisture-density tests were made.

Samples for density tests were obtained by sampling through a hollow stem auger with a standard 3" Shelby tube. Measurements were taken carefully before and after each push. The length of the sample in the tube was carefully measured and compared with the depth sampled to determine compression, if any. The relatively undisturbed sample was used to compute the volume of the hole and the material density.

Low density, dry soils were found throughout the area of undisturbed soils to a depth of about 20 feet. As previously stated the soils were found to be primarily silt (A-4) and silty sand (A-2-4) with minor stringers of sandy gravel (A-1-b). The moisture-density-blowcount relationship of the SPT tests also showed low-density soils, or a potential for soil collapse.

Further information was gained by means of the double consolidometer test. The difference in the void ratio of identical soil samples run in a wet and dry state indicated a potential collapse of about 31 inches throughout a 20 foot depth. This indicated that the soils below the existing scab patches may undergo additional collapse in the future. However, moisture-density tests made directly below some of the roadway sags indicated a marked improvement in density and moisture over the natural soils or soils that have apparently not received excessive moisture since deposition. This improvement reaches to a depth of about 10 feet below the original ground surface. The moisture is 100% higher and the density has increased as much as 10 pounds per cubic foot.

Several modes of stabilization were considered including: sub-excavation, flooding the area with water, injection of chemicals, deep plowing or ripping and wetting to above optimum moisture, ponding reverse sand drains, vibroflotation, and dynamic compaction.

Sub-excavation and replacement under standard moisture-density requirements obviously would solve the problem of densification, provided that the depth of treatment is thick enough to bridge the unstable soils. Since cost estimates are easily calculated for sub-excavation, this mode of stabilization as an experimental exercise was not needed. The main point of the experiment was to compare the efficacy and cost of other methods of stabilization with sub-excavation.

Flooding the area with water and allowing collapse to take place, which was apparently used successfully on the San Luis Canal in California, was chosen as one of the methods for the experiment in spite of some known difficulties that would exist during construction. Some of these difficulties are: the anticipated time required for traffic diversion to a detour, controlling the collapse area on mainline construction (a power plant, several businesses and a buried gas line lie adjacent to the highway), lack of information relating to time constraints, the enormous volume of water required for flooding, the difficulty of monitoring the settlement and the probable need to surcharge the section to assure subsidence.

Chemical injection was rejected because no chemical showed any promise of penetrating the relatively impermeable soils.

Eventually, four methods of stabilization were selected which showed potential success for partial or complete stabilization of the soils, as well as keeping costs competitive with excavation and replacement. These were: 1) deep plowing or ripping and wetting the soils to above optimum moisture, and compacting with a heavy vibratory roller, 2) place reverse sand drains with ponding facilities and supply sufficient water to initiate collapse, followed by vibratory roller compaction; 3) vibroflotation or vibroreplacement, using the European-developed vibroflot probe; and 4) dynamic densification using the impact of a free-falling weight for compaction, another European development.

Deep plowing was picked as a "poor boy" process. Little chance of complete success was expected, but it would provide useful information relating to: recompacting loose soils, driving moisture into undisturbed soils by rolling, initiating, and propagating collapse without soaking, and bridging collapsible areas by providing an upper densified zone.

Reverse gravel drains were selected as the next most complex effort to offer the maximum in surface treatment, which could reasonably be done by a general contractor without specialized equipment. The design would allow: complete wetting to the base of the collapsible zone, observation of infiltration rates, natural collapse tendencies with increased water availability, and the effect of a vibratory roller on wetted soils at various depths.

Vibroflotation and dynamic compaction are specialized methods of densification requiring some degree of proprietary equipment and knowledge of application. These methods would be restricted to demonstration by companies with substantial experience in each paticular technique.

Vibroflotation applies a long probe or vibroflot hanging from a crane. The probe vibrates from an eccentric weight driven by an electric or hydraulic motor mounted inside. Powerful water jets are located near the tip and along the probe. The vibroflot gradually is lowered through the layer of soil to be treated by means of vibration, water jets and its own weight. By raising and lowering the probe and, when necessary, feeding gravel into the hole and working it into the surrounding ground, the soils are compressed and reworked into a higher density state.

Dynamic compaction is a more direct approach to densification. It requires lifting a very heavy weight to a substantial height repeatedly and dropping it on the surface. Penetration, hydrostatic pressures in the groundwater, and blow counts are monitored carefully. The size and configuration of the weight, the height and drop frequency, and the number of blows per impact area can be varied to produce the desired results.

It was felt that both vibroflotation and dynamic compaction could produce the desired results; but, since neither method had been used on collapsible soils, there was no basis for bidding, no proof that either process would work and, little chance that accurate, low, and competitive bids could be obtained on a production project. Therefore, the important part of the experimental project was to inform the companies involved of the performance of their equipment on this type of soil and to lessen the risk and promote cost savings in excess of the experimental costs.

An early problem in the design of the test section project was in deriving a test method for quality control. Since the primary consideration was density, this was the obvious means to use. However, measuring density at depth, using conventional tools, would require opening relatively large holes throughout each test section, which could severely alter the test results. Almost all of the methods considered for establishing natural ground data had the potential to alter the results.

Eventually, outside the proposed test plots, a number of test holes were drilled with a 6 5/8 inch hollow stem auger. Densities were taken as in the preliminary investigation, and controlled length Shelby samples were taken ahead of the auger bit, at five-foot intervals, in the drilled holes, to a depth of 30 feet. Moisture, density, gradation, and Atterberg limit tests were run on all Shelby samples. Two-inch disposable point penetrometer tests were run near the drill holes, and later in the test plots to establish that the test plots had comparable soils. This prevented unnecessary disturbance of the ground in the test plots. It was also hoped that disposable points could be used to monitor density changes during the compaction process. This was done successfully only in the dynamic compaction plot which required no water. The Shelby tube sampler became the primary method for measuring density changes upon completion of the compaction process in each test plot. These results were compared with densities taken by the same method in the buffer zones.

The double consolidation tests of the collapsible soil samples served a dual purpose on this project. First, they indicated that the soils were indeed collapsible, and provided information for making an estimate of the maximum amount of settlement that might be expected. The calculations showed that about 31 inches of collapse could be expected upon total saturation. Secondly, they provided minimum target densities that were needed at various depths to prevent further collapse.

Figure 2 shows the minimum target density relationship to natural ground or pre-treatment density. The target dry density varied from 85 PCF near the surface to 91 PCF at a 30-foot depth. The dynamic effects of traffic vibration on the soils were not used in calculating the minimum target densities. The double consolidation tests also showed that a load of 9 tons per square foot would be required to collapse these soils in their natural state of moisture.

In addition to the testing procedures above, in the dynamic compaction plot, the contractor used a pressuremeter; screw plate, and dilatometer to monitor the change in density.

The test plots were laid out in a linear fashion along the proposed alignment of the east frontage road and future detour for mainline construction, from station 163+00 to station 174+00 (figure 1.) This placed them close to the northbound traffic from Albuquerque to Santa Fe and limited traffic control was required.

The dynamic compaction and deep plowing plots were 50 x 100 feet, the other two were 70 x 120 feet. A 100-foot buffer zone was left between plots to prevent mechanical or other disturbance from affecting the end results.

A number of ideas were explored for a bid letting, including separate and combined lettings through standard State Highway Department contract letting procedures, and through the State Purchasing Agent. Finally, in March 1980, after considerable effort the State asked for bids to build the experimental section as one unit, allowing the contractor to sublet the vibroflot and dynamic compaction operations to appropriate subcontractors. The bidding was restrained and the low bid was 268 percent above the engineer's estimate of $131,672.00. After reconsideration all bids were rejected.

At this time, for bidding purposes, the experimental section was split into separate units. The deep plowing and reverse gravel drain plots were made a part of project I-025-4(57)240, a multimillion dollar frontage road project which was to precede mainline construction. Arrangements were made for the other two plots to be bid through the State Purchasing Agent. The dynamic compaction as a State Purchasing Agent (SPA) purchase order, and the vibroflotation as a SPA parts and labor contract. Project I-025-4(57)240 routinely went to contract and was awarded to the Broce Construction Company of New Mexico, Tucumcari, New Mexico.

Difficulties related to the unique nature of the experimental project continued to plague the SPA orders and each was cancelled again because of failure to meet Federal requirements. Eventually, on December 16, 1980, bids were opened, and the vibroflotation part of the experimental project was awarded to the Vibroflotation Foundation Co. of Pittsburg, Pennsylvania, and the dynamic consolidation part was awarded to the

Hayward Baker Co. of Odentown, Maryland. The award of the contracts was further delayed by bonding and contract licensing problems, but these were eventually resolved.

DEEP PLOWING
TEST PLOT 4

The purpose of this part of the experiment was to determine the effective depth of stabilization caused by deep plowing, raising the moisture content to above optimum, and compacting the soils with a heavy, steel drum, vibratory roller. Construction of the test plot was let to contract along with the frontage road construction and Broce Construction was the successful bidder.

Prior to construction, six rod soundings were made using 2 1/4 inch disposable points on a 1 3/4 inch (AW) rod. The SPT method of cathead, pull rope, 140 pound hammer and 30-inch drop was used to drive the points. In-place densities were made in two locations between the test plots at three-foot intervals to a depth of 30 feet. Ground elevations were taken throughout the test plot in order to check total settlement after construction.

The initial work began on this plot in January 1981. The ground was plowed to a depth of 3 feet with a D-9 Caterpillar tractor and a single tooth ripper. Eight thousand gallons of water were used to raise the moisture of the plowed ground to 5 percentage points above optimum.

No specific soaking period was required for this plot; however, a drying period of about one week was required before it was dry enough for the vibratory roller operation. A heavy vibratory, steel drum roller made two sets of four passes over the test area and tests were made as shown below.

For additional control, before rolling and after wetting, two rod soundings, using the disposable points, were made in the test plot. Additional soundings were made within two feet of these soundings after each set of passes. These were compared with the original wet soundings to determine if any improvement had been accomplished. Final rod soundings were made within three feet of the first (dry ground) soundings to determine the effect of the construction process, Figure 4-2.

After construction, in-place densities were taken through a hollow stem auger in four locations, at three-foot intervals, to a depth of 30 feet.

The rod soundings showed a significant decrease in the soil strength on each set of roller passes. In fact, the greatest decrease in strength occurred between the first and second set of passes. So far there has been no feasible explanation for these results and they are considered to be inconclusive.

The results in the initial and final rod soundings seem somewhat more reliable, and compare favorably with the density tests. The final tests show an average decrease in the blow count of about 33 percent to a depth ranging from 4 to 10 feet. Below this, down to 30 feet, the rod soundings were about the same.

Post-Construction testing showed a significant decrease in density in the upper 10 feet of soil -- as much as 10 pounds per cubic foot. Below 10 feet there was no significant change.

Ground elevations were checked upon completion of construction and there was no apparent lowering of the surface.

It is concluded that this method of treatment is unacceptable since no substantial improvement in soil strength or density was shown by tests made with the disposable point penetrometer or the Shelby tube sampler.

PRE-WETTING
TEST PLOT 3

The purpose of this part of the experiment was to show the effective depth, lateral extent, and the collapse potential that total or partial saturation would have on the soils of the area. Construction of the test plot was let to contract along with test plot four and the frontage road construction. Broce Construction was the successful bidder.

As on test plot 4, prior to construction, six rod soundings were taken over the area and in-place densities were taken in the buffer zones. Initial ground elevations were taken to compare with final ground elevations upon completion of construction. Pre-wetting was to be accomplished by the use of reverse gravel drains.

Work began on this plot in January 1981. Eighteen eight-inch diameter holes were drilled on 20-foot centers to a depth of 15 feet. Each hole was fitted with a two-inch, perforated P.V.C. pipe and back-filled with gravel. A dike, ten feet in diameter and two feet high was built around each hole, and water was ponded inside each dike for ten days. A period of ten days was set for water infiltration from data obtained from in-situ permeability tests. A total of 238,000 gallons of water was used during the ponding period.

After 13 days the contractor began rolling this plot with a vibratory steel drum roller. As on plot 4, rod soundings were made with disposable points, after wetting and between sets of roller passes. Four sets of roller passes were made and disposable points were driven at 10 locations.

Six final rod soundings were made near the original (pre-treatment) soundings. Eleven in-place densities spaced five to seven feet apart were taken along the center of the treated plot between three of the gravel drains. These tests were located 5 to 12 feet from the drains. Tests were made at five-foot depth intervals to a depth of 30 feet. Both dry density and moisture content were compared with the control samples taken prior to treatment.

As expected the rod soundings taken before treatment were considerably higher than those taken after treatment, except for the gravel zones.

This held true for all of the six sites tested within the plot. This data shows the possible depth and extent of water infiltration into a collapsible soil by the use of reverse gravel drains. The apparent loss in soil strength is attributed to water loosening or dissolving the cementing bonds between the soil particles of the dry soils.

The average moisture content increased from 200 to 300 percent throughout the 30 foot depth of soil for a radial distance of eight feet from the gravel drains. Radially, from eight to 12 feet, the average moisture content increased from 30 to 50 percent to a depth of 30 feet.

Within an eight-foot radius of the drains the soil density increased an average of seven pounds per cubic foot to a depth of 7 feet. From 7 to 12 feet very little improvement was gained. From 14 to 16 feet the density increased 3 1/2 PCF, and at each successive five-foot interval the average density increased one PCF. No apparent increase was shown at 30 feet, nor was there any increase within the 30 foot section of soil beyond the 8 foot radius.

The final ground elevations varied from 6 inches to as much as 24 inches lower than the original surface at completion of construction.

After construction of this test plot, a grid pattern of five prints was tamped using the dynamic compaction technique. Craters six feet deep were created after 8 drops from a height of 38 feet.

During construction of the test section and frontage road, near the test section, the contractor's water line leaked in several places along the frontage road. The soils collapsed from 20 to 30 inches around these leaks, and where the road was involved the collapsed areas were repaired before final construction. One collapsed area remains in the right-of-way fence line 60' right of frontage road centerline station 184+50. This collapse was recently examined and the following data collected:

Depth (ft.)	Density (PCF)	% Moisture	% Pass 200 Si.	Soil Type
3.6- 5.0	90.0	10.5	47	A-4
5.0- 6.5	89.5	9.3	38	A-4
10.0-11.5	85.5	11.4	55	A-4
15.0-16.5	89.7	22.1	75	A-4
20.5-21.5	72.3	21.7	63	A-4
25.0-26.5	66.2	24.7	78	A-4
30.0-31.5	105.1	17.6	62	A-4

Collapsed area about 30 feet in diameter water line leaked about 15 days. These soils are comparable to those below the test section and the data clearly shows an improvement in the soil moisture and density.

It appears water should have been ponded over the test plot for a longer period, perhaps an additional five days, for better moisture distribution. The gravel drains should have been placed closer together and the vibratory roller seems to have had some favorable effect to a depth of seven feet. However, pre-wetting did not increase soil density enough to warrant its use in successfully treating the collapsible soils of this area.

VIBORFLOT COMPACTION
TEST PLOT 2

The primary purpose of this part of the experiment was to determine the total effectiveness of vibroflotation in improving the density of the collapsible soils along this section of Interstate 25 and to investigate the effects of: 1) spacing between compaction probe positions, 2) depth of penetration, 3) probe withdrawal procedures, and 4) wetting agent in the jetting water.

The invitation to bid was divided into four items: Mobilization, vibroflot compaction, backfill material, and water. Since there was a variable number of units used in each of the bid items except for mobilization which was bid as a lump sum, the initial bid price was subject to change depending on the number of units actually used.

The Vibroflotation Foundation Co. of Pittsburgh, Pennsylvania was awarded the contract on December 16, 1980 at an estimated cost of $42,998.50.

Six disposable point penetration tests (rod soundings) were made prior to and after treatment to determine the effect of the vibroflot compaction. Final Shelby tube samples were taken for moisture-density tests and compared with initial tests made in the buffer zones. Ground elevations were recorded before and after treatment to measure subsidence. The final rod soundings were inconclusive because the points encountered the coarse-grained gravels that had been pushed into the walls of the probe holes. The ground elevations were of no value because the construction method displaced a greater volume of soil than the expected collapse.

In the initial planning for this section, three probe spacings were used - six feet, nine feet, and 12 feet apart. This is the distance between the probe sites positioned in an equilateral, triangular pattern. During construction it was found that probing a six-foot pattern was difficult to achieve. The vibroflot "walked" away from the previously compacted areas. For this reason compaction in the 6' pattern areas were changed to seven-foot centers, which were easily probed.

Twenty feet and 34 feet were used as penetration depths. Roughly half of the test plot was compacted to 20 feet, the other half was compacted to 34 feet. Twenty feet was considered to be a reasonable depth of the collapse-prone soils because there was some indication that the upper limit of the ground water table had reached this depth sometime during the past. The existing ground water table is 60 feet below ground surface. Thirty-four feet was used arbitrarily to determine if the deeper soils had collapsed during periods of ground water saturation. This was also the length of the vibroflot provided.

Three probe withdrawal procedures were used. The first, "30 seconds/foot vibro method" is used in conventional compaction of clean sand. The vibroflot is withdrawn in one-foot increments and suspended for 30 seconds at each increment, while backfill (gravel) is fed down the hole. The second, called "2-foot lift stone columns," consists of lifting the vibroflot three feet, backfilling with gravel, then repenetrating the backfill one foot. Repenetration forces the backfill laterally into the walls of the cavity. The third procedure, "one-foot lift stone columns," is similar to the second but involves lifting the probe two feet and repenetrating one foot. There was no time requirement for either stone column procedure, but they were completed as quickly as possible.

A wetting agent was to be added to the jet water for the last three compactions to determine if wetting the soil could be accelerated. Seven 32 ounce boxes of "Cascade" brand non-sudsing dishwashing detergent were added to the sump which contained about 24,000 gallons of water.

Construction equipment was mobilized in late March 1981. It consisted of a vibroflot probe, a 50-ton truck crane, a front-end loader with a two-yard bucket, a 6-inch high pressure pump, and miscellaneous support equipment.

Water was obtained from a privately owned well, located across the Interstate about 3/4 mile to the southwest. The well pump was used to move the water about 4000 feet through a 6-inch hose to a sump, constructed at the west end of a culvert, located under I-25 about 1000 feet north of the test section. The vibroflotation, high-pressure pump pulled the water from the sump and pushed it through a single 2½-inch hose to the manifold on the crane, which distributed the water to the vibroflot jets. One and three quarter to three-inch, well-rounded gravel obtained from a gravel pit about one mile south of the test plot, was used for backfill.

The compaction operation started by penetrating the very hard, silty sand with the vibroflot. This was accomplished by turning on the two tip jets to full flow. The combination of the jetting action and vibration broke up and displaced the soil and the vibroflot sank under its own weight. Due to the hardness of the soil the vibroflot was frequently lifted, and dropped for better penetration. Once full depth penetration was attained the withdrawal, or compaction, phase began. Where the "30 second/foot" procedure was used, the tip jets were turned almost completely off, backfill was piled on the surface around the vibroflot where it settled through the water in the annular space around the vibroflot, and the vibroflot was lifted 1 foot each 30 seconds. Where the stone column procedures were used, the holes were first surge flushed by raising the vibroflot out of the holes then quickly repenetrating them. Stone was then added and the vibroflot raised and "lowered" according to the "lift" thickness specified.

Records were made of penetration time, compaction time, and number of loader buckets of stone for each compaction point. The loader buckets are only an approximate measure of the backfill because the volume of loose rock varied between loads, and variable amounts of rock were left on the surface. Water quantity was measured at the well.

A total of 74 compaction points were completed. The total linear footage was 1,970.

The contractor completed the work as bid bid on December 16, 1980. The work was done in substantial compliance with the specifications included in the contract conditions. The project was completed in a period of less than 10 working days. Completion date: April 7, 1981.

A total of 44 test holes 30 feet deep were drilled in the test plot. Samples were taken at 5-foot depth intervals for density moisture tests. Tests were made on 254 samples. Most of the holes were placed at the center of the triangular-shaped sites of the compaction point locations. The results of highly erratic tests were not used for averaging density. Three test holes were used to measure the possibility of moisture migration after using the wetting agent in the jet water in sub-section "q". Holes were placed 4 ft, 8 ft., and 12 ft. from the point of treatment.

Penetration rates were slow compared to routine sand densification projects. The overall average rate of penetration was about 3.33 feet per minute. This rate would have been significantly greater had 2 hoses been used between the high pressure pump and the manifold, and had a weighted follower tube been used. The vibroflot pulled 140 to 170 amps during penetration.

Withdrawal rates were different for the three compaction procedures. The 30 second/ft. vibro procedure, and the 1 foot lift stone column procedure took about the same amount of time - about 2 ft/minute. For the 2 foot lift stone column procedure, the average withdrawal rate was much faster - about 3 ft/min. Amperage generally built to 70-90 amps for the 30 seconds/foot vibro procedure and to 100-120 amps during re-penetration of the backfill for the other two procedures.

The amount of backfill placed in each hole seemed to be a function of the amount of time taken to withdraw the vibroflot. The least number of loads were used for the 2' lift stone column procedure which was the fastest. A total of 1332 tons of backfill were used in the 74 holes. It is estimated that 10 to 15 tons of this quantity were left on the surface. Hence, the average backfill usage was about 1320 tons/1970 ft. = 0.67 ton/ft.

Two hundred eighty-seven thousand, six hundred gallons of water were pumped from the well. An unmeasured but significant portion of it ran off the test site. In all but one hole, in sub-section "a," some jet water returned to the surface during jetting and withdrawal of the vibroflot. On that single hole, water was lost between depths of 25 and 34 feet. The water tended to pond over the site and eventually breach the low dikes and run off to the culvert to the north. The surface soil absorbed water very slowly. The shallow ponds would not seep into the soil over night. Over the weekend, however, the site did dry up. This could have been a result of evaporation.

No direct indication of the effectiveness of the various procedures were observed on the surface of the test plot during construction, however the fact that the ground accepted so much backfill material was an indication of improvement of the in-situ material. That a loader was operated on the wet surface of the completed portions of the test plot also indicated improvement of the in-situ soils.

A study of average values for the post-treatment densities was made for comparison with pre-treatment densities. There seems to be a marked improvement over the entire test plot with the closer probe spacings, 6 and 7 feet, being generally better than the 9 and 12 foot spacings. The mode of withdrawal didn't seem to affect the results. Densities at the 10, 16, and 26-foot elevations are higher than the 20 and 22-foot elevations in sub-section "a" through "j," and the density generally decreased at the 30-foot elevation. Target densities were reached for both the 6 and 7 foot spacings. Some of the tests show an apparent increase in density where there was actually no increase in soil moisture. There was no noticeable moisture change in sub-section "q" where the detergent was added to aid in moisture migration.

There is no doubt that the natural ground conditions were improved throughout the test plot. The stone columns at each compaction point represent a very substantial improvement. However, whether the soils actually collapsed enough along the outer perimeter of the columns remains to be seen, particularly between 9 and 12-foot probe spacings. Further collapse may be seen after saturation by surface runoff.

With some modification of the probe spacings, the vibroflot method of compaction may be an acceptable method for treating collapsible soils.

Probe Layout

Sub-Section	Spacing	Depth	Withdrawal Procedure
a	9'	20'	30 sec/ft vibro
b	9'	20'	2 ft/ lift stone columns
c	9'	20'	1 ft. lift stone columns
d	9'	34'	30 sec/ft vibro
e	9'	34'	2 ft lift stone columns
f	9'	34'	1 ft lift stone columns
g	12'	34'	30 sec/ft vibro
h	12'	34'	1 ft lift stone columns
i	12'	20'	30 sec/ft vibro
j	12'	20'	1 ft lift stone columns
k	6'	20'	30 sec/ft vibro
l	6'	20'	2 ft lift stone columns
m	6'	20'	1 ft lift stone columns
n	7'	34'	30 sec/ft vibro
o	7'	34'	2 ft lift stone columns
p	7'	34'	1 ft lift stone columns
q	12'	34'	1 ft lift stone columns (wetting agent in water)

Figure 2-2

DYNAMIC COMPACTION
TEST PLOT I

The purpose of this part of the experiment was to determine the effectiveness of Dynamic Compaction in collapsing the low density soils along this portion of Interstate 25, and to determine the print spacing, height of drop, number of drops, and attitude of the weight required to best accomplish the results.

Two bids were received for construction of this plot. Menard, Inc. of Monroeville, Pa. bid $89,400.00. Hayward Baker Co. of Odentown, Md. was awarded the contract for a low bid of $58,600.00, $25,000 for mobilization and 0.14/ft. ton of compaction estimated at 240,000 ft. tons, equaling $33,600.00. This part of the test section was let to contract on December 16, 1980.

Six disposable point penetration tests were run in January 1981 by SHD personnel. These were to be compared with final penetration tests to show the apparent increase, if any, in density.

Pressure-meter, dilatometer and screw plate tests were made previous to, during, and after construction, to show any apparent increase in soil density caused from the compactive effort.

Since the ground-water table lies about 60 feet below the surface, there was no need to measure the change in pore-pressures.

The number of drops for densification were to be determined by a decrease in the weight penetration of 0.2 foot per drop at each print in grid one.

It was assumed that if the final soil densities were above the target densities shown from the double consolidometer tests, then satisfactory consolidation would have been achieved.

The 100 by 50 foot test plot was divided into eight grids of equal size, 25 x 25 feet. A plan was made for tamping each grid where the height of drop, attitude of the weight, number of drops, and spacing of the print pattern could be varied to show which method would accomplish the best results.

The contractor moved a suitable crane and a 6'x6'x3' high, 20 ton, steel weight to the job site on May 7, 1981. On May 11, the contractor completed rigging his crane and test dropped the 20 ton weight 5 times outside the test plot from 35 feet. On this date, Clyde R. Anderson with the aid of the SHD drilling crew performed the initial pressure meter test on Grid 3-1. On May 12, the crane production tamping got underway and two more initial pressure meter tests were made at grids 6-3 and 7-3.

With the exception of grid 3 and 5, the print pattern was a 12 foot square and drops were made at the corners and middle of each square. On grid 3, the drops were made at the corners of a 10 foot square and one in the middle. On grid five, three drops were made in the north portion and one was made in the middle. Random drops were made at the top, or sides of the grids. A total of 44 prints were tamped and the number of drops varied from three to eight. The height of drop varied from 19 to 38 to 75 feet.

Production tamping went rather routinely. Clyde Anderson made secondary and tertiary pressure meter tests and returned home on May 15, 1981. Schmertmann and Crapps, Inc., Consulting Geotechnical Engineers ran the screw plate and dilatometer (DMT) tests.

The screwplate test was not successful because of gravel in the soils and the DMT was usable only when taking special precautions. Some success was gained in the dilatometer test and it showed measureable and significant soil improvement as a result of the dynamic compaction, to depths of 25.6 feet.

A five print tamping procedure was carried out in the gravel drain plot. Again, drops were made at the corners and middle of a 12 foot square. The weight was flat and the drop height was 38 feet. The average crater depth was 6.4', considerably more than any comparable grid print in the dynamic compaction plot. This was obviously the result of extra moisture.

The contractors testing and tamping operation was completed on May 22, and the area was cleaned up and graded 5-26-81 prior to removal of equipment. A total of approximately 120 cubic yards of sandy borrow was used to level the craters.

Upon completion of construction of test plot 1, from 5-26-81 through 6-5-81, twenty-seven holes were drilled with a hollow-stem auger, and 118 controlled length Shelby tube samples were taken at approximately or about five foot intervals to a depth of 30 feet for moisture density tests. Holes were drilled immediately below the center print of all eight grids, immediately below several corner prints and randomly at the middle of three and five print patterns.

The density curves show a marked improvement in the soils to a depth of 20 feet. The target densities derived from double consolidometer tests were exceeded in most of the grids, and the average post-density values usually were near or above 90 PCF.

The disposable point penetration record (rod soundings) showed a substantial increase in soil strength up to depths of 30 feet.

Among the different soils stabilization techniques used in this experiment, Dynamic Compaction is rated as the most successful. The natural ground conditions were improved up to depths of 30 feet throughout the test plot. Natural ground conditions improved greatly between the prints as well as immediately below the drops. Only one part of the experiment looks questionable, and that is where the drop height was reduced to 19 feet.

It is believed that the Dynamic Compaction method of stabilization will be competitive with sub-excavation and replacement, where stabilization is desirable to depths of 10 to 20 feet.

ALGODONES TEST SECTION
DYNAMIC COMPACTION PLOT 1
Drop Record

Date	Grid No.	Print No.	Drop Height	Weight Attitude	No. Drops	Crater Depth	Appx. Print Spacing
5-14-81	1	1NEC	38'	Flat	7 *	3.9'	8.5'
"	N.E. Cor.	2NWC	75'	"	8	4.5'	"
"		3CEN	38'	End	4 *	5.2'	"
"		4SEC	75'	Flat	7	4.2'	"
"		5SWC	72'	End	6	5.3'	"
5-14-81	2	1NEC	38'	Flat	8	3.33'	"
"	N.W. Cor.	2NWC	"	"	"	2.76'	"
"		3CEN	"	"	" *	3.36'	"
"		4SEC	"	"	"	3.59'	"
"		5SWC	"	"	"	2.96'	"
"		6EC	"	"	"	2.97'	"
5-22-81	3	1NEC	"	"	8	3.12'	7.1'
"	N. Cen.	2NWC	"	"	"	3.26'	"
"	E. Side	3CEN	"	"	" *	3.43'	"
"		4SEC	"	"	"	3.54'	"
"		5SWC	"	"	"	3.51'	"
"		6WC	"	"	"	3.22'	"

Date	Grid No.	Print No.	Drop Height	Weight Attitude	No. Drops	Crater Depth	Appx. Print Spacing
5-15-81	4	1NEC	19'	"	8	2.97'	8.5'
"	N. Cen.	2NWC	"	"	"	2.69'	"
"	W. Side	3CEN	"	"	" *	2.95'	"
"		4SEC	"	"	"	2.72'	"
"		5SWC	"	"	"	2.72'	"
5-19-81		6NC	38'	"	"	3.48'	"
5-21-81	5	1NEC	75'	"	3	3.54'	"
"	S. Cen.	2WOC	"	"	"	3.03'	"
"	E. Side	3CEN	"	"	" *	2.74'	"
"		6NC	"	"	"	2.62'	"
5-13-81	6	1NEC	19'	End	8	4.5'	8.5'
"	S. Cen.	2NWC	"	"	"	4.48'	"
"	W. Side	3CEN	"	"	" *	4.70'	"
"		4SEC	"	"	" *	4.66'	"
"		5SWC	"	"	"	6.41'	"
5-14-81	7	1NEC	75'	Flat	3	2.78'	"
"	S.E. Cor.	2NWC	"	"	"	3.01'	"
"		3CEN	"	"	" *	2.29'	"
"		4SEC	"	"	"	2.66'	"
"		5SWC	"	"	"	2.44'	"
"		6CWS	"	"	"	2.65'	"
"		7CNS	"	"	"	2.64'	"

Date	Grid No.	Print No.	Drop Height	Weight Attitude	No. Drops	Crater Depth	Print Spacing
5-15-81	7	1NEC	75'	"	3	1.77'	"
"	S.E. Cor.	2NWC	75'	"	"	2.34'	"
"		3CEN	75'	"	" *	1.87'	"
"		4SEC	75'	"	" *	2.15'	"
5-18-81	8	1NEC	38'	End	7	7.77'	"
5-13-81	S.W. Cor.	2NWC	38'	"	"	5.27'	"
5-18-81		3CEN	38'	"	" *	5.48'	"
5-13-81		4SEC	"	"	"	5.49'	"
"		5SWC	"	"	"	6.47'	"
5-18-81	RGD	1NEC	38'	Flat	3	7.05'	"
"	(Reverse	2NWC	"	"	"	6.46	"
5-19-81	Gravel	3CEN	"	"	"	5.30	"
5-18-81	Drain	4SEC	"	"	"	6.23	"
"	Plot)	5SEC	"	"	"	5.83	"

*Final density test taken immediately below drop. Other densities were taken at the center of 3 drop and 4 drop patterns.

SUMMARY

The experiment on four techniques for stabilizing collapsible soils was successfully completed in the Algodones area in 1981. The four methods used in the experiment and the order in which they were built were: Deep Plowing, watering and rolling (Test Plot 4); Pre-wetting, by use of reverse gravel drains and rolling with a heavy steel-drum, vibratory roller (Test Plot 3); Vibroflot Compaction, Vibro-replacement of the soils creating stone or gravel columns (Test Plot 2); and Dynamic Compaction, densifying soils by dropping a heavy steel weight (Test Plot 1).

Excellent cooperation was given by all parties concerned in this project. All of the contractors were substantially within the specifications during construction, and many of the unknowns about the treatment of collapsible soils were answered, particularly for the four trial methods.

The data produced will be of interest to contractors and engineers in estimating costs and selecting the most desirable, and economical method for densifying soils to depths below the capabilities of conventional equipment.

Some of the test plots will need further observation and possibly more testing for some time before final conclusions can be drawn, possibly for as long as 3 to 5 years.

Deep Plowing Test Plot 4
The Deep Plowing technique used on Test Plot 4 was the least effective of all of the trial methods in the experiment. Virtually no improvement was shown in soil density throughout the 20 to 25 foot zone of collapsible soils. The disposable-point rod soundings were of as much value here as they were in any of the plots that were wetted. The data produced with the disposable-point during rolling seems to be somewhat questionable when compared with the pre-construction and post-construction rod data; however, it confirms what construction people have been reporting for years, i.e. continued rolling with the vibratory roller may destroy the initial density gained by a limited number of passes. Elevations taken before and after treatment in this plot showed no apparent lowering of the surface.

In any event this method of treatment for collapsible soils of more than a few feet in thickness proved unacceptable.

Pre-Wetting Test Plot 3
The Pre-wetting, reverse gravel drain technique showed some promise of success, although it was deemed unsuccessful in the experiment. The final elevations in test plot 3 varied from 6 to 24 inches lower than the original ground. Because of the collapse that took place below the

contractors leaking water line, adjacent to the test section, and the fact that some collapse is shown by final elevation measurements, it is strongly believed that had the gravel drains been placed closer together and a longer ponding period been allowed for saturation, then the soils would have collapsed from their own weight. Or, use of a sprinkler system to soak the soils over a longer period would result in sufficient collapse to stabilize these soils without gravel drains. Further research is needed to make a fair prognosis of the value of this method of treatment.

The disposable-point penetration test was of some value in evaluating the moisture changes in the soils when pre-wetting. Where the blow count dropped off, it is clear that the soil moisture increased. The Shelby tube densities, however, proved to be the best test for quality control and for measuring soil changes. It is clear now that more density tests could have been made during the boring process of the gravel drain installation. This would have given valuable information in the final evaluation of the effectiveness of all of the techniques used.

Vibroflot Compaction Test Plot 2

The Vibroflot Compaction method of soils stabilization produced more improvement in the soils than either of the above; although the final effect is still somewhat inconclusive. The 6 and 7 foot spacing of probe points and implacement of stone columns produced the best results in density improvement in the inter column areas. The contractor believes that some subsidence may occur in the inter-column areas in the future, particularly where the probe spacings were 9 to 12 feet apart and soil density did not change appreciably. This remains speculative, for it is not known nor is there any way of measuring what the effectiveness of stone columns are below an asphalt mat. There was no noticeable change in the final surface elevation of this plot. This is understandable because the stone columns displaced a greater volume than was expected from collapse.

The use of the disposable-point for measuring final soil parameters was abandoned for this plot because of encountering the stone columns. Average density changes were measured successfully, although with difficulty, by use of the Shelby tube. For better control, more preconstruction tests could probably have been made without radically changing the final results. Boring a hole immediately below a probe point would not have altered the final results appreciably, at all.

Dynamic Compaction Test Plot 1

The Dynamic Compaction technique of stabilizing collapsible soils in this area was the most effective. The soil density improved considerably over most of the test plot, not only immediately below the weight drops, but in the inter-print areas as well. Target densities were exceeded in most instances. The results were questionable in only one grid where the 19 foot drop was used. The final elevation of the test plot wasn't measurable because it was pitted with craters left from the weight drops. Had the area been leveled and compacted it would have shown an average reduction of more than 24 inches in elevation.

All of the SHD's proposed techniques for quality control were good. Only on this test plot was the disposable-point penetration test completely successful. The gain in soil strength was clearly indicated in the final test and the Shelby tube method of sampling for density tests was very successful here. It is probably the best method to use for soils of this type.

The pressure-meter tests showed an improvement in the deformation modulus. More tests should probably have been made with the pressure meter for a better evaluation.

The screwplate and Dilatometer tests were unsuccessful because of gravel within the deposit.

Cost Analysis

In so far as costs are concerned, few conclusions can be drawn from the experiment. Construction of the Dynamic Compaction and Vibroflot Compaction plots were apparently bid rather high because of estimated mobilization costs; however, both contractors rented part of their equipment locally. The unit cost of the experiment was as follows:

Plot No.	Compaction Technique	Mobilization Cost	Price/ yd^2	Total
IV	Deep Plowing	Neg.	4.25	2,362.00
III	Pre-wetting	Neg.	11.14	10,399.00
II	Vibroflot Compaction	15,320.00	34.72	47,731.99
I	Dynamic Compaction	25,000.00	60.48	58,600.00

On a job immediately north of the experimental section, where 19,000 square yards of ground were set up for Dynamic Compaction, the lump sum bid was $95,000.00 or $5.00 per square yard. On future higher production jobs this unit cost will probably be reduced. The cost estimate of complete excavation and replacement to depths of 10 feet is approximately twice this amount, about $9.00 per square yard and there would be no guarantee of complete success in this particular area unless some improvement is gained to depths of 20 feet.

Given the choice of whether to compact the soil during the initial stage of construction or leave it for future maintenance, there seems to be little doubt that the tax payer or user would benefit more from compaction during the initial stage of construction. Besides the wear on vehicles and inconvenience of a wavy ride, maintenance costs over a five year period are estimated at about $7.00 per square yard. This would include scab-patching and leveling with a minimum thickness overlay of about 2½ to 3 inches of asphalt pavement.

Since at least four of the techniques used in this experiment seem to be within the range of success, the New Mexico State Highway Department has decided to let the contractor choose his method of stabilizing collapsible soils. Specification requirements should be written around the premise that after compaction or densification, the soils will support any additional or existing load regardless of artificial or natural moisture changes.

This paper is condensed from the final report which will be available shortly. Persons desiring a copy of the complete report should write or call the Materials Laboratory, New Mexico State Highway Department, P.O. Box 1149, Santa Fe, New Mexico 87501, phone 505-983-0550.

ENGINEERING GEOLOGY, RELOCATION OF STATE HIGHWAY 91

CLIMAX MINE AREA, SUMMIT COUNTY, COLORADO

By

John B. Ivey
Amuedo and Ivey, Inc.

and

Jerome B. Hansen
Colorado School of Mines
Research Institute

1.0 INTRODUCTION

During 1976 the proposed increase of the tailings from Amax Corporation's Climax molybdenum mine necessitated relocation of a 4.17-mile stretch of State Highway 91 north of Fremont Pass in Summit County, Colorado. The geologic conditions along the proposed alignment from Sta. 35+00 to the northern terminus (Sta. 220+00) were mapped, described, and evaluated by geologists from Amuedo and Ivey, under contract to Centennial Engineering, Inc., design engineers for the project. This paper summarizes the findings of the study and the recommendations made to that firm. Although the project has been in operation for several years this paper is written essentially in the present tense, and in the form that the report was submitted.

1.1 Description of Project Area

The project area lies along the eastern slope of the valley of Tenmile Creek between Clinton Creek on the south and Graveline Gulch on the north (Fig. 1). Elevations range from just above 10,200 feet, to just below 11,500 feet msl.

The Tenmile Creek Valley trends roughly northeastward through mountainous terrain. Most of the valley is comprised of fairly steep walls produced by glacial erosion and deposition. The surface of the area consists mainly of Pleistocene glacial till and outwash and shallow Quaternary colluvium which overlies bedrock slopes.

The major streams in the study area, Clinton Creek (now dammed), Mayflower Creek and Humbug Creek all drain into Tenmile Creek which flows northeastward into Dillon Reservoir, approximately 12 miles from the project area. These streams generally run close to bank-full in the late spring due to the large quantity of melt water derived from the winter snow accumulation, which is in excess of 100" in a normal season. Many small, steep-graded, intermittent streams also drain the valley sides.

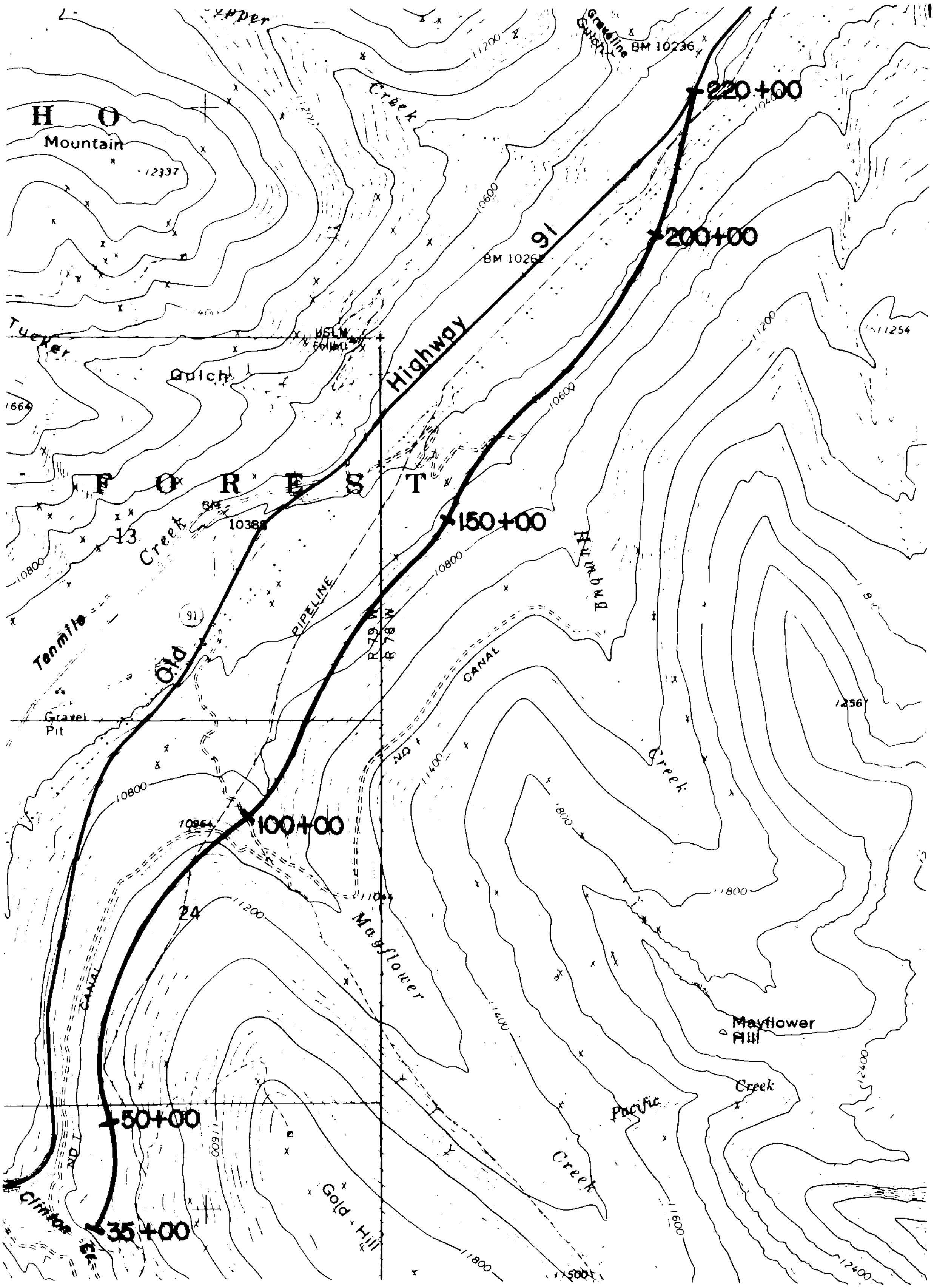


Figure 1. Highway 91 Relocation

In addition to these streams, the alignment is dotted with many small springs and seeps. Some of these flow throughout the summer but most of them appear to slow or stop by late summer and early fall, the driest portions of the year. Many small, boggy and marshy areas are associated with the seeps, springs and small ephemeral streams.

Most of the study area is densely wooded with mature stands of lodgepole pine. Wet, poorly drained areas support thickets of willows and lush, broad-leafed ground cover. Some drier areas have developed into mountain meadows covered with grasses and shrubs.

Before construction of the present alignment, access to the site was by means of unimproved dirt roads and construction trails. Primary access was by graded dirt roads along Clinton, Mayflower, and Humbug Creeks and the No. 1 canal (Fig. 1). Each of these roads was accessible via the old Highway 91.

1.2 Purpose and Scope

The purpose of this project is to collect the geologic information necessary for the engineering design of the highway, for slope recommendations, and for mitigation measures in areas of present or potential land failure.

Several alignments have been proposed for the highway at various times. The first one was established by Amax Corporation and was staked in 1973. The Colorado Division of Highways developed a different alignment and surveyed 100-foot stations along it in 1974 and 1975. Profiles for these stations were developed from topographic maps produced in 1968. Surveyed elevations were established at random intervals along the centerline of the alignment staked by the Division of Highways.

The scope of the work involved a review of data previously assembled by Climax Molybdenum Company and its consultants, and by the Colorado Department of Highways. This data included surficial and bedrock mapping, seismic surveying, monitoring bore-hole inclinometers and information acquired from prospect pits. Since most of these data had not been prepared specifically for the highway project, it was necessary to do engineering geologic mapping, and additional geophysical work, test pit excavation, and test drilling. The evaluation of data from all sources provided the input to the design engineers.

1.3 Project Constraints

This project is unique in that options for alternate alignments are literally restricted within a few feet in the most critical areas. Each of the alignments which have been staked is within a few feet of each other, and of the final one. Because alignment options are so severely restricted in the engineering sense it is apparent that the geology will control the project. Hence, the engineering geologic investigations have concentrated on a field review of known problem areas, and the identification of previously unrecognized problems.

The poorly exposed geology makes it impossible to determine precisely in a cursory review where failure and distress in natural and manmade features can be expected. Certain areas are identified from direct observation as being prone to failure; we are confident that other areas will be identified during construction. It is apparent that the extent to which geologic conditions are masked by vegetation and surficial material will force many engineering decisions in the field during construction.

1.4 Methods and Procedures

An office photogeologic review of the area was made on aerial photographs and detailed field mapping of the project was conducted between July 28 and September 21, 1976. Mapping was done on topographic bases at a scale of 1"=200' supplied by Climax Molybdenum Co. Copies of these maps annotated with the known regional geologic data were used as bases for detailed field mapping. A control network along the alignment was surveyed by Climax in the most densely vegetated areas to provide reference points for our detailed geologic mapping by Brunton compass and tape/pace traverses, and by triangulation methods.

Specifications were developed for four test holes which were drilled by the Colorado Department of Highways under contract to Climax. Three holes were drilled along a line normal to the projected alignment on centerline and to either side, just south of Mayflower Creek where bedrock was shallow. A fourth hole was drilled in the alluvium of the creek on centerline. Samples were collected of the unconsolidated materials in each hole at irregular intervals, and bedrock was cored to a few feet below grade. All samples were described and cores were logged for lithology, structure and Rock Quality Designation (RQD).

A test pit program was designed and executed jointly by our firm and R.W. Thompson, Inc., the project soils engineering consultant, to test surficial material and depth to bedrock. Eleven pits were dug with a backhoe, descriptions were made and samples were collected for testing by Thompson. This work supplemented a test pit program which had been conducted by Climax in late 1975 (1c.).

1.5 Acknowledgements

This project was undertaken by the authors, and by J.L. Hynes, then employed at Amuedo and Ivey. Each of these persons participated in the field and office aspects of the project.

During the project considerable help was provided by Messrs. Warren Alloway, William Klauber and Vaughan Surface, all of Climax Molybdenum. Their time in the office, and the time Messrs. Alloway and Klauber spent in the field with our personnel is gratefully acknowledged.

Acknowledgement is also given to John B. Gilmore and Alan Eastwood of the Colorado Department of Highways for conferring with our personnel and for their cooperation in seismic investigations and test drilling done by the Department.

Finally, thanks are due to Mr. H.L. Noble, a special soils engineering consultant to Centennial Engineering, Inc. and especially to Mr. Richard F. Sparlin, Vice President of that firm for his permission and encouragement to publish the results of this investigation.

2.0 GENERAL GEOLOGY

The geology of this area has been of interest since before the beginning of this century because of the potential for the occurrence of economic mineral deposits. The Climax molybdenum mine about four miles south of the new alignment, is the most important mineral operation in the area. The general geology of the area is shown in Figure 2.

2.1 Stratigraphy

The major bedrock units in the area consist of the Late Paleozoic sedimentary Minturn and Maroon Formations which overlie Precambrian metamorphic gneisses and migmatites. Several Tertiary intrusives have invaded both the sedimentary and metamorphic rocks of the area. All bedrock units are overlain by glacial till or post-glacial unconsolidated deposits ranging in thickness from only a few feet to more than 100 feet.

The Precambrian metamorphic rocks are highly altered metasedimentary gneisses and migmatities of the Idaho Springs Formation. The principal minerals in these rocks are quartz, feldspar, biotite and hornblende. Feldspar crystals are preferentially aligned and clusters of quartz crystals are readily observed in the rock. Banding in the gneisses shows evidence of both flowage and failure (2,4). Pronounced foliations are easily observable in some of the migmatites and essentially absent in others.

The Minturn and Maroon Formations are characterized by red-to-gray arkoses, siltstones and mudstones (2). Many of these rocks contain abundant mica, often in layers along bedding planes. The most extensive exposures of these rocks are found in the deep roadcut just south of Clinton Dam. Intensively metamorphosed and mineralized quartzites and arkoses which are broadly classified as metasedimentary rocks are locally associated with the sedimentary rocks. Due to poor exposures, the relationship of these rocks to the Minturn and Maroon Formations could not be precisely determined.

The Tertiary igneous intrusives in the area generally are quartz monzonite porphyries and the only variations between them are in the accessory minerals and grain sizes of each (2). Some poorly developed contact metamorphism may be found in the host rocks adjacent to the intrusives.

Morainal material and associated outwash and alluvial fans overlie the bedrock along Tenmile Creek and all of its major tributary streams, Clinton, Mayflower, Humbug, and Copper Creeks. Where these deposits are well drained and have a stabilizing vegetation mat they appear relatively stable. Where exposed to excessive moisture, these deposits locally become highly mobile and unstable at existing slope angles.

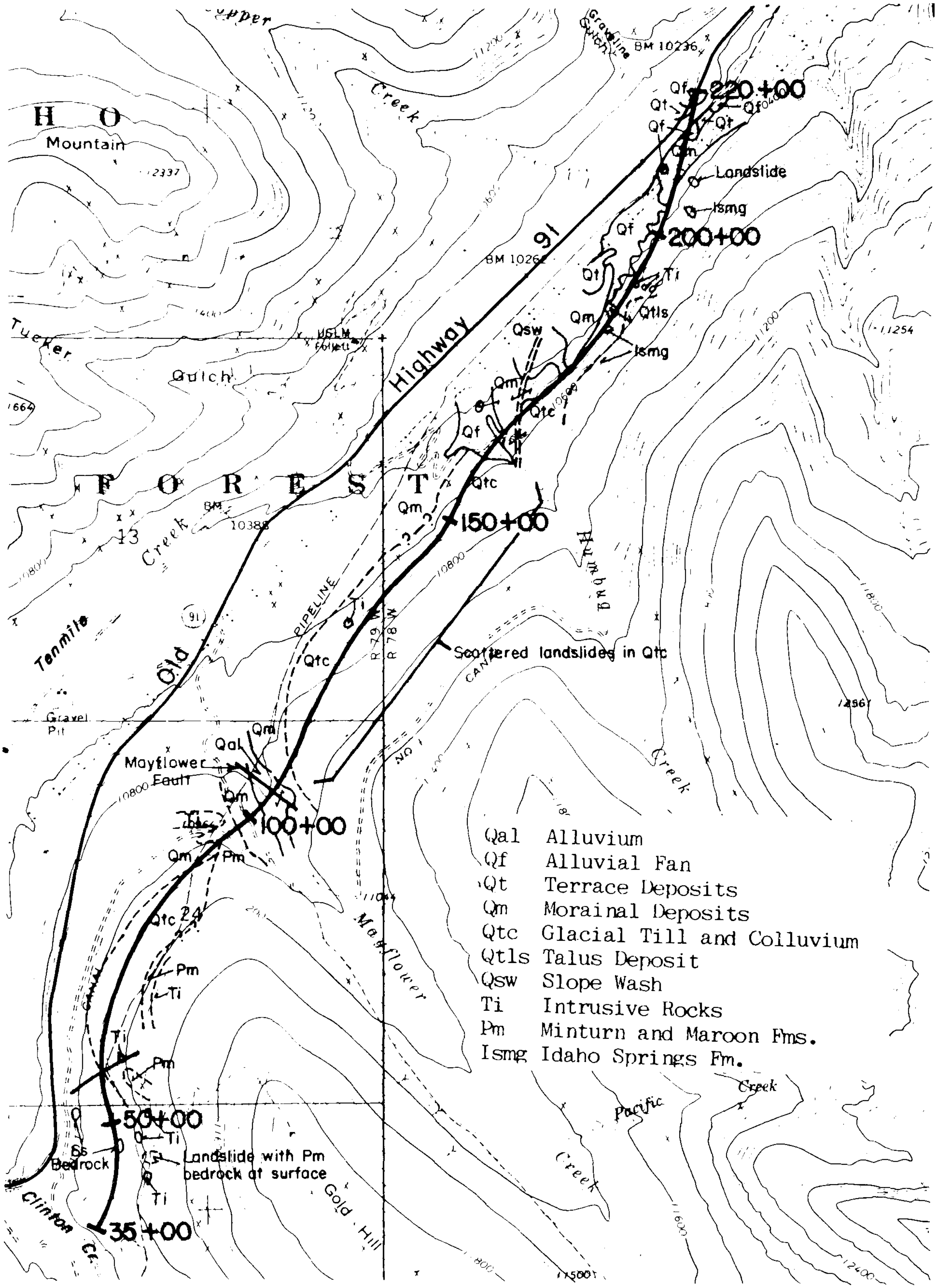


Figure 2. General Geology

8236

Above the major valley floors glacial material interfingers with slope wash, talus, colluvium and residuum. Much of this material is also unstable at the surface, and many small-scale landslides, slumps and stepped surfaces have developed.

2.2 Structure

The sedimentary and metasedimentary units in the area have dips ranging from 25° to 70° to the west and northwest in the few locations where they could be measured. Mapping of the area done by Climax (1a.) has shown major faults which intercept the alignment in three places. Surface evidence of this faulting was not observed during the detailed mapping program in the narrow strip studied.

The bedrock exposed in the study area was extensively jointed and fractured. The limited rock exposures and the small number of data points available, precluded any meaningful analysis of the relationship between the joints and fractures observed and the regional structural patterns. Much of the fracturing may be due to relatively recent ice action and may not persist to significant depths.

3.0 ENGINEERING GEOLOGY

The engineering geology investigation included determination of depth to bedrock, engineering properties of the rocks and unconsolidated sediments, hydrology, slope stability, and a discussion of problem areas. The physical characteristics and stability of unconsolidated materials were quantified by the project soils engineering consultant.

The generally poor and limited exposures of bedrock along the alignment made impossible the direct observation of structural features which control cut-slope stability. A zone approximately 1000 feet wide (500 feet either side of the alignment center-line) was mapped first on aerial photographs and then on the ground.

3.1 Depth to Bedrock

Depth to bedrock at various locations throughout the alignment was determined by drilling, backhoe test pits, rock outcrop projections and seismic surveys. Overburden thickness varies from zero to over 100 feet. Most of the alignment is within areas underlain by glacial till, and colluvium (Fig. 2). The alignment crosses thick deposits of glacial moraine and outwash where Clinton, Mayflower and Humbug Creeks enter the valley of Tenmile Creek. Significant zones of shallow or exposed bedrock occur within the mapped area generally about 250 to 500 feet east or upslope from the center line. An exception to this is on the south side of Mayflower Creek where rock exposures were mapped on both sides of the center line.

The depth to bedrock, and the configuration of the bedrock overburden interface are of particular concern in the so-called "tenderloin" area. Here, the depth to and configuration of the bedrock surface was estimated from surface mapping, drill-hole data, seismic investigations, and test pit excavations.

The bedrock configuration was formed largely by the glaciation of the valley of Tenmile Creek. The northward-moving ice left a relatively smooth bedrock wall which slopes northwestward on the eastern side of the valley. On this wall lies a vegetated cover of glacial material, weathered rock and colluvium of variable thickness and distribution. Irregularities in the bedrock are found locally, and these are generally oriented parallel to or nearly parallel to the valley. Local bedrock highs may form buttresses against the downslope movement of unconsolidated overburden.

The overburden thins to the southeast, uphill from the alignment. This thinning was observed in prospect and assessment pits. Generally within 1000 feet east of the alignment the overburden is from five to ten feet thick.

3.2 Engineering Properties of Geologic Units

3.2.1 Sedimentary Rocks

The Minturn and Maroon Formations consist primarily of sandstones, siltstones and mudstones which if undisturbed, should provide a good sub-base for the roadway. These rocks appear to be well jointed in the few exposures nearest the alignment (see Fig. 3). Alteration of these rocks by weathering and mechanical means probably has produced zones or areas of weakness. Mechanical weathering will produce small fragments which subsequently could act as unconsolidated material, and chemical weathering and alteration could produce zones or layers of plastic, easily deformed and easily eroded material. The presence of mica in these rocks may contribute to their instability in local areas.

An initial concern with bedded sedimentary rocks was instability in cut slopes where bedding or joint-planes would be undercut. Unfortunately the exposures of these rocks are limited in extent and were too far from the alignment to be of value for any definitive analysis in the areas of interest. The high degree of fracturing observed in the few exposures along the alignment and in the test core drilling imply that most of the sedimentary bedrock encountered at or above final grade should be rippable.

3.2.2 Igneous and Metamorphic Rocks

The igneous intrusives of Tertiary age in the study area are quartz monzonite porphyries which should be relatively sound and stable in their unaltered state (see Fig. 4). If these rocks are encountered in a weathered and(or) fractured condition at grade along the alignment they could cause stability problems.

The metasedimentary rocks of Precambrian age observed in the study area are primarily gneisses and migmatites. One exception is a fairly common metaquartzite associated with the sedimentary units described earlier. The gneisses and migmatites display poor- to well-developed foliation. This foliation represents zones of weakness along which movement or failure could occur under load or where undercut in excavations. The metaquartzite appears to be highly fractured and jointed. It is estimated that it might need to be stabilized in cuts but should provide sound sub-base beneath fills.

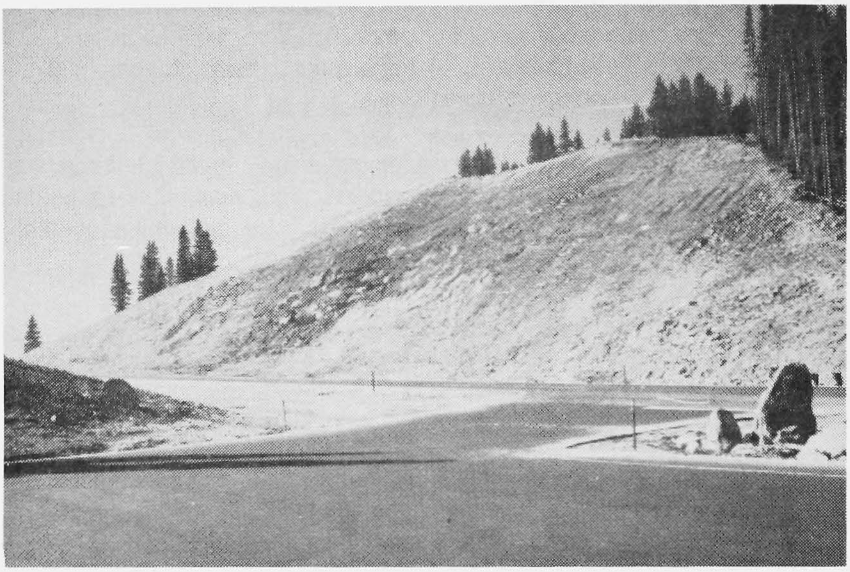


Figure 3. Exposure of Minturn-Maroon Formations in west side of road cut on south side of Clinton Dam.

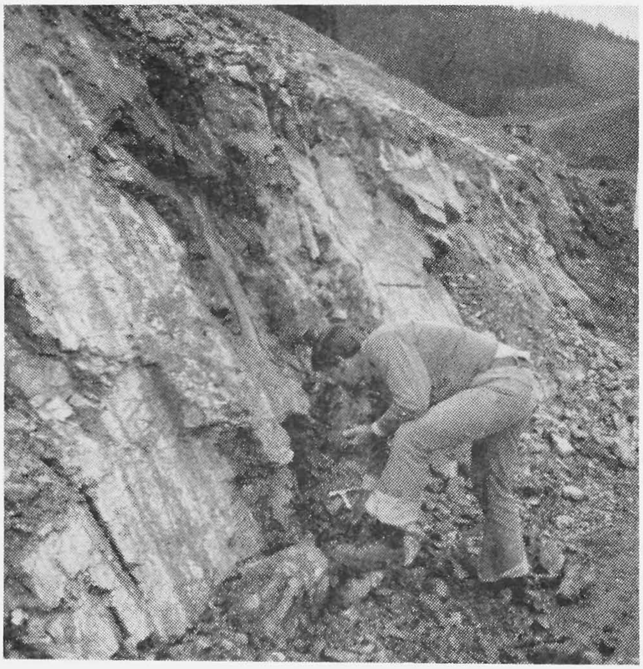


Figure 4. Exposure of Tertiary intrusive rock in trench dug on south side of Mayflower gulch.

Exposures of intrusive rocks in outcrops or test pits along the alignment were dug easily with a backhoe because they were weathered and well jointed (see Figs. 5 and 6). These rocks should prove to be rippable close to the surface where frost action and weathering have weakened them.

Seepage zones associated with metasedimentary rocks in test pits and where rock was close to the surface indicated that, if highly jointed, metaquartzite should provide a permeable zone or channel which might cause water-related problems such as piping or slumping. It is estimated that the highly fractured metaquartzites should be rippable.

3.2.3 Unconsolidated Surficial Deposits

Unconsolidated surficial deposits along the alignment consisted of glacial till and outwash, colluvium, talus, residuum, alluvium and landslide and slump material. All of these deposits can be moved with bulldozers, scrapers, and backhoes, although some boulders may be too large for equipment other than bulldozers to handle.

3.2.3.1 Till, Outwash, Colluvium, Talus, and Residuum

Glacial till, colluvium, talus and residuum occur as an irregular mantle of unconsolidated material overlying bedrock throughout most of the area. Till also occurs in morphologically distinct landforms similar to moraines. This material is in general, poorly sorted, and unstratified (see Figs. 7 and 8). The mineral composition and grain-size distribution vary considerably. Much till, colluvium, and talus are found in naturally oversteepened environments and may have been temporarily stabilized under certain critical conditions of moisture and(or) vegetation.

In moist, poorly drained areas, such as the "tenderloin" (Stas. 115+00 to 157+00), which exist along the alignment, disruption of the surface material in any way is likely to cause failure (see Fig. 9). Engineered drainage and slopes appear to be the most readily applied corrective measures in this situation. Additional engineering design (i.e. retaining walls and armored slopes) may be needed in specific locations. Removal and replacement of some of this material is suggested as a remedial measure if zones of high clay content or organic matter are encountered during construction.

It is anticipated that a number of talus trains, which were buried under several feet of organic material, will be uncovered during construction. The greatest likelihood for this is in the general area north of the "tenderloin" to Sta. 185+00.

Areas of outwash and alluvium should present little problem either in construction or maintenance. These deposits are fairly permeable and stable and can be easily excavated, if necessary. Because of their mode of deposition these materials should be well compacted and should provide good foundation for fill with little or no settling except where local lenses of organic matter or clay occur. Neither of these

Figure 5. Backhoe in operation on pioneer road in "tenderloin" area.

Figure 6. Backhoe pit in jointed and weathered Tertiary intrusive rock, "tenderloin" area.

Figure 7. Glacial till exposed above Borrow Area B. Material on left (west) has slumped; note light-colored scar marking uphill limit of slump.

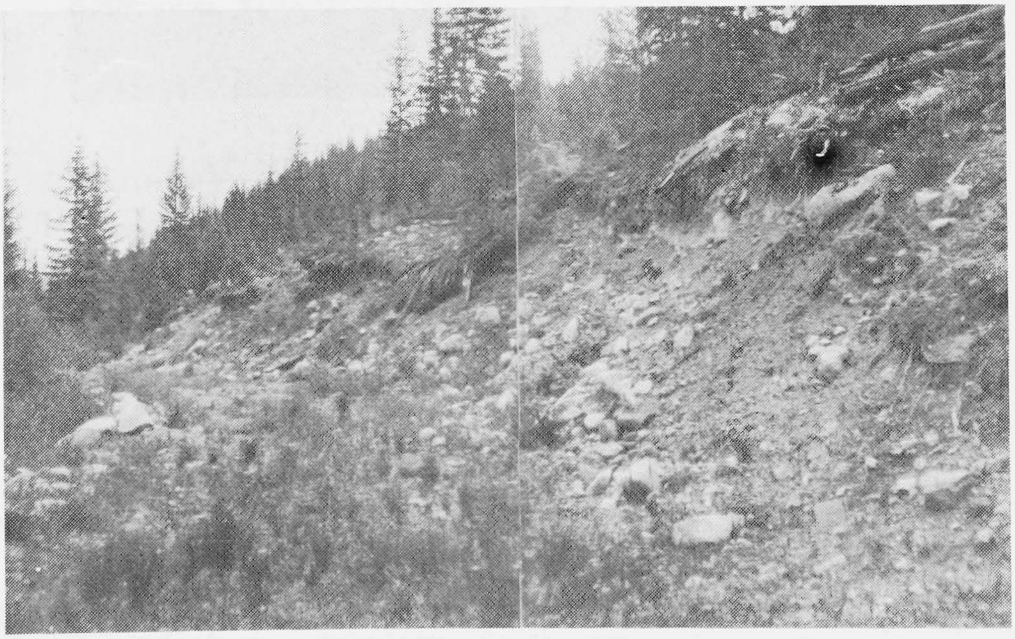


Figure 8. Glacial till exposed in old drill site road at about Sta. 164+50.

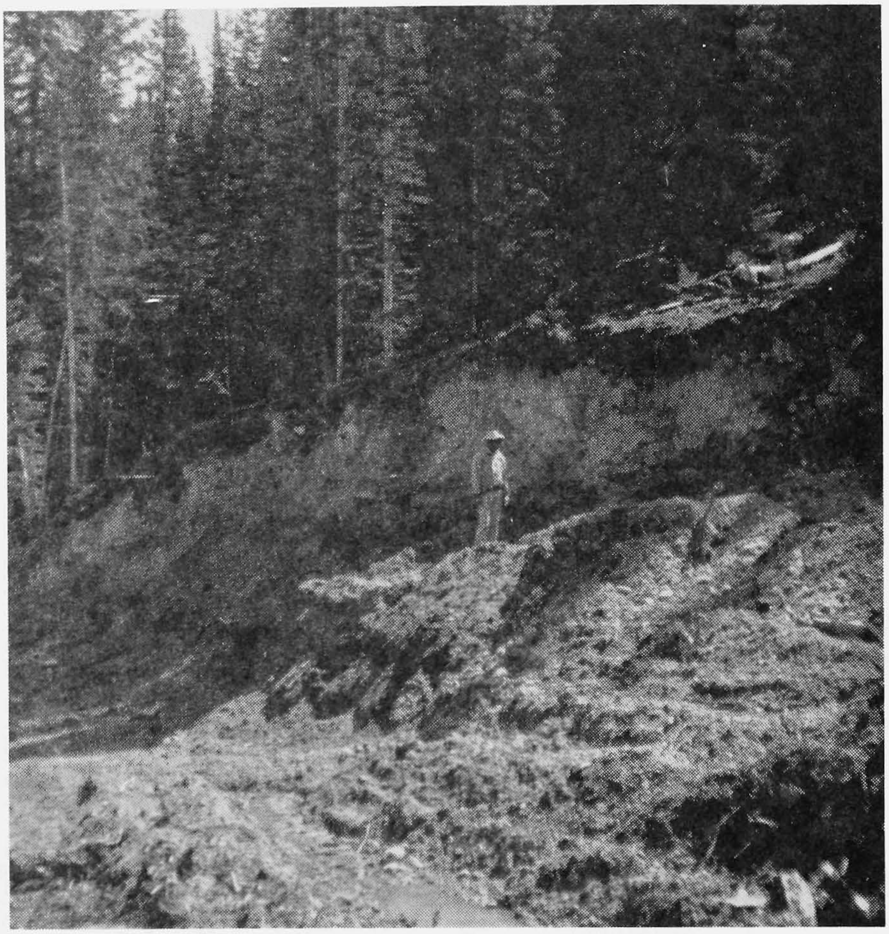


Figure 9. Slump in cut for drill road at about Sta. 124+50. After cut was made slumping apparently occurred in several episodes.

undesirable materials was observed during the field mapping or in the test drilling conducted in Mayflower Creek. Sufficient quantities of organic material found at the surface or encountered above grade possibly can be used for revegetation after stripping and stockpiling.

Residuum consists of soil and decomposed rock weathered in place. This material interfingers with the till and colluvium and is more predominant where the glacial deposits thin in an uphill direction. The residuum is generally thin, granular and contains a large amount of organic matter, silt and clay. Some of this material is very clay-rich and plastic, although "fat" clays were not observed.

3.2.3.2 Landslide and Slump

Landslide and slump materials occur in several places along the alignment. Locally this material is estimated to be in temporary equilibrium and extremely sensitive to disruption. Failure in these materials may be reactivated as a result of construction activities. Most of the failure features observed during the detailed mapping were assumed to be due to small, shallow-seated shear planes. The optimum time for failure to occur is most likely during the high-moisture period in late spring when thaw penetrates to the frost line. Figure 10 is a picture of a failure that occurred during the field work as a result of heavy rains between August 6 and 10, 1976. It is estimated that much of this type of material will have to be removed or drained and reinforced to insure the integrity and safety of the roadway.

In the "tenderloin" area soft ground was produced as a result of disruption of wet unconsolidated material. This has resulted in hazards to heavy equipment during construction of pioneer roads and during the test pit program. Similar conditions may also exist in the large boggy area around Sta. 165+00.

3.3 Slope Stability

Engineering geology factors regarding slope stability are described here in qualitative terms primarily on the basis of geological factors. Quantitative aspects of slope stability are covered in a companion soils engineering study.

3.3.1 Bedrock

Slope stability in the highly jointed and fractured sandstones, siltstones and the metaquartzite will be determined by the strike and dip of rock encountered during excavation.

In metamorphic basement rocks the orientation of jointing and foliations will determine the relative cut stability at various slope angles. Cut-slope stability in the unaltered intrusive rocks will be controlled largely by the orientation and dip of joints. Generally, these rocks should be sound and should be stable in fairly steep cuts. The presence of intrusive rock may also add an element of strength to the rocks it has intruded.

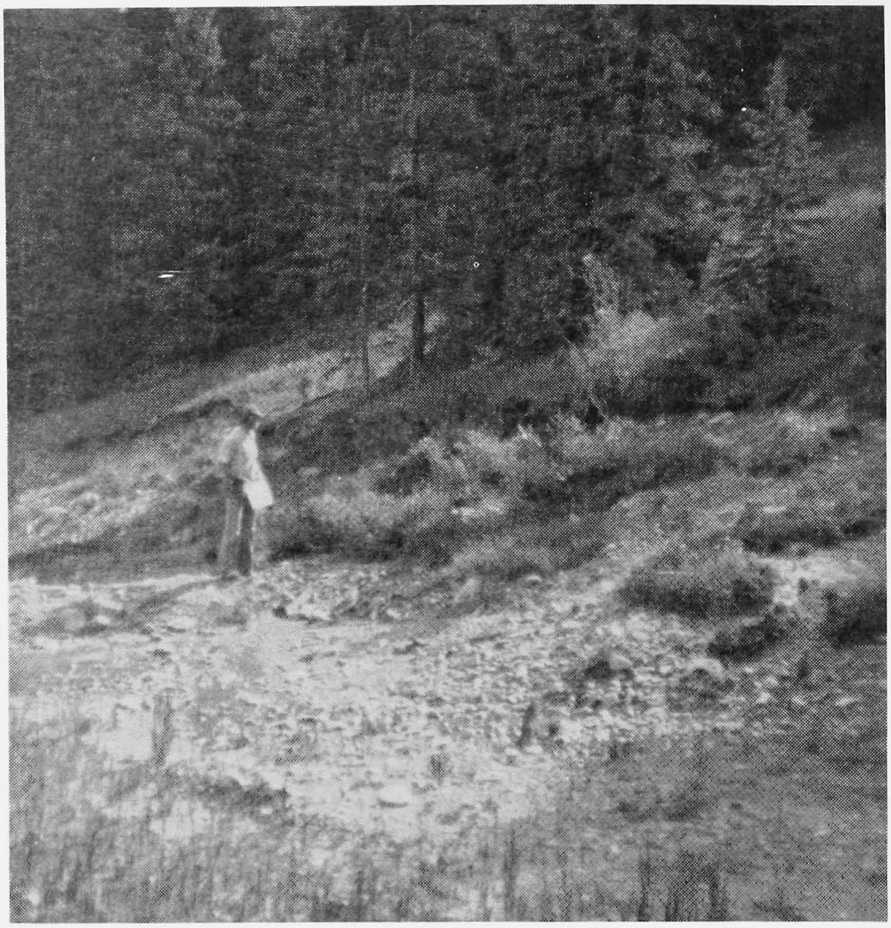


Figure 10. Small landslide or slump that occurred between August 6 and 10, 1976 in morainal material at about Sta. 110 on the north side of Mayflower Creek.

3.3.2 Unconsolidated Deposits

Many small cut slopes (generally less than six feet) in unconsolidated deposits which were observed in the project area appeared to be relatively stable even at near-vertical angles, whereas larger cuts appeared to be unstable even at fairly shallow slopes of about 2:1. Cut slopes much shallower than 2:1 are not practical on many portions of the alignment because they will not "daylight" for several hundred feet. Unconsolidated deposits are particularly subject to erosion and to mass failure.

3.4 Hydrology

Annual precipitation in this area is approximately 20 inches with the majority of it falling as snow in the winter months (8). A great deal of this moisture runs off during the late spring while the ground is still frozen and impermeable. The remainder evaporates, and percolates downhill in the shallow subsurface as vadose water through the unconsolidated material along the steep slopes of the valley.

Much of this vadose water stays in the unconsolidated materials until, 1) it reaches a stream course where it becomes surface water, 2) it is intercepted by small surface depressions, or 3) it encounters impermeable barriers and feeds bogs, springs and seeps. Water reaching the surface in or just above cut slopes, or under fill sections, can cause serious problems. A preliminary site-specific drainage program was developed for the "tenderloin" by the design engineers with geological inputs. This drainage program was partially implemented in the fall of 1976 in an effort to dewater some problem areas and facilitate construction in the spring of 1977.

Based on the field information acquired it is concluded that the water table is well below the ground surface, but that perched water tables can be expected throughout the area.

3.5 Problem Areas

Problem areas are identified in order to define conditions which should be mitigated in design (Fig. 11). It should be noted that even slight changes in the alignment may alter the relative effect of these problems on the roadway, eliminating certain site-specific problems and introducing others not noted in this study.

3.5.1 Borrow Area B

This borrow area is developed in a thick wedge of glacial till northeast of Clinton Creek from Sta. 35+00 to Sta. 54+00. This deep cut had been made to provide fill for the Clinton Creek dam prior to the time this investigation began.

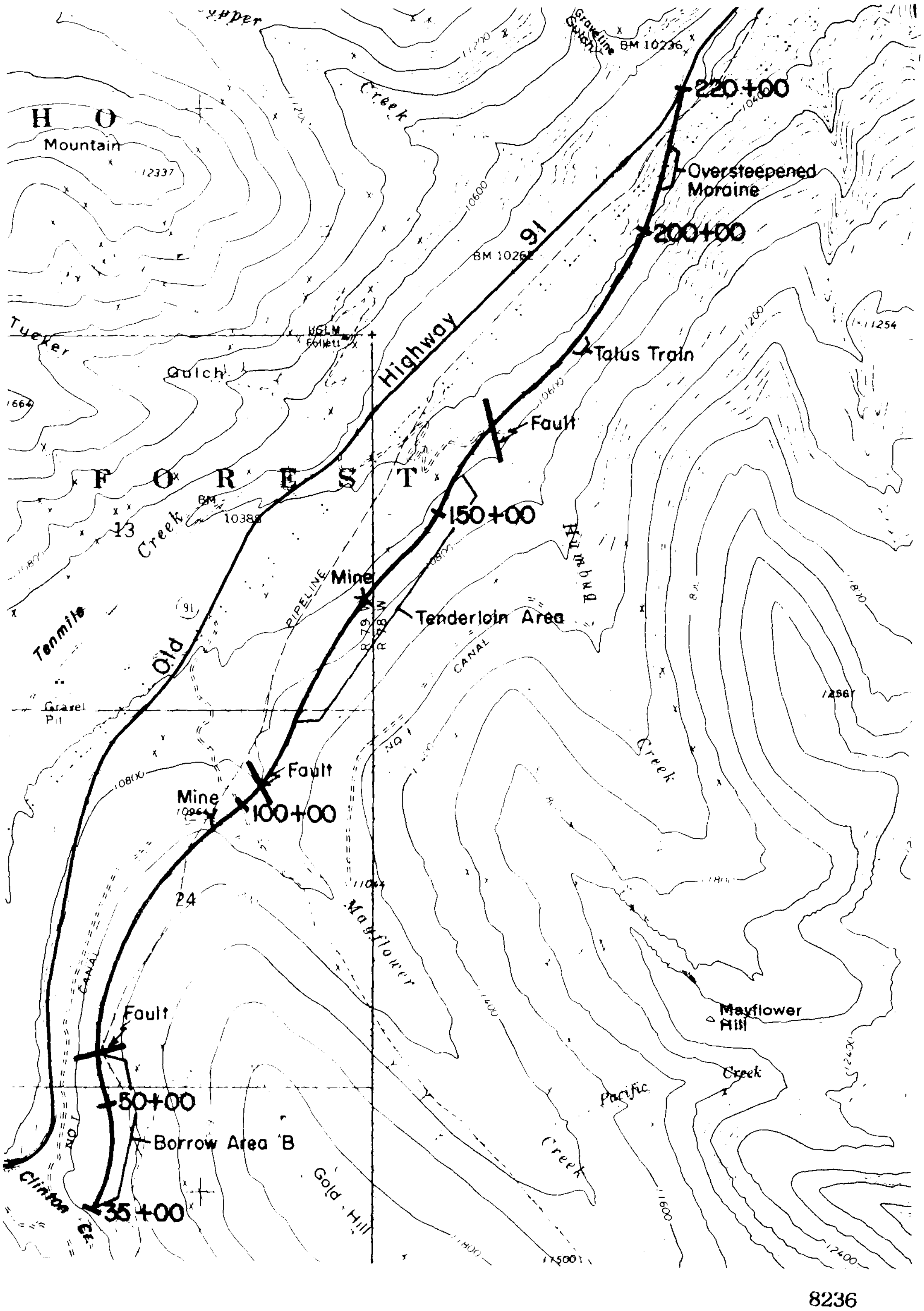


Figure 11. Problem Areas

The higher slope on the right of the alignment has undergone failure since it was first opened in the fall of 1974, and was still failing in September, 1976 (Fig. 12). Failure ranged from incipient to advanced during the period of field observation (July 28 thru September 21, 1976). The most important factor associated with this problem appeared to be the extreme instability of the clay-rich unconsolidated material in the presence of excess moisture. Figure 13 is a generalized plan of Borrow Area B as of early September, 1976. Note the common association of water with slumped areas.

Even as the expected period of low ground water flow approached there were still many moist areas and several active springs and seeps visible in and just above the cut. The most significant of these springs was a network which issued from a slump mass possibly associated with an old collapsed mine adit above the cut, between Stas. 42+00 and 43+50, 350-450 feet right. This water drained into an open, unlined trench which ran northward parallel to the top of the cut for 140 feet and then down the face of the cut to a plastic drain pipe. The unlined portion of the surface drainage system, and ruptures in the plastic pipe operated as a recharge system to the cut area.

A lined cut-off trench is recommended along the abandoned gas-line bench at the top of this cut to divert water southward from about Sta. 45+00 into the impoundment behind Clinton Dam. Northward, from about Sta. 45+00 to Sta. 53+00, this trench would protect that part of the cut which shows signs of incipient failure. At its north end the trench could be drained under the new highway into one of several natural drainage courses between Stas. 54+00 and 56+00.

Based on detailed mapping of the cut, and an indicated depth to bedrock of 86' below grade determined from seismic data, the project soils engineer undertook an investigative program in this cut, to determine a stabilization scheme.

3.5.2 Fault Zones

Three faults or fault zones (4) cross the projected alignment. Unstable ground conditions exist in all three of these areas but no direct correlations could be demonstrated between this instability and the faults. The absence of any definitive surficial evidence of these faults implies that they are inactive.

The trace of the unnamed southernmost fault is shown in Figure 2. Downhill from where it crosses the alignment the fault is marked by numerous seeps and springs, hummocky ground, and small-scale failures in cuts, fills and natural slopes. This area is apparently part of an old landslide. If this fault is exposed in bedrock cuts, it could be a source of water and an engineered drainage plan may be needed to correct any adverse water problems.

The Mayflower fault intersects the alignment as shown in Figure 2. A zone 300 feet wide on the northeast side of the fault includes a large area of spongy, hummocky ground with many seeps, leaning and "pistol-butted" trees and signs of surface instability. A series of pronounced

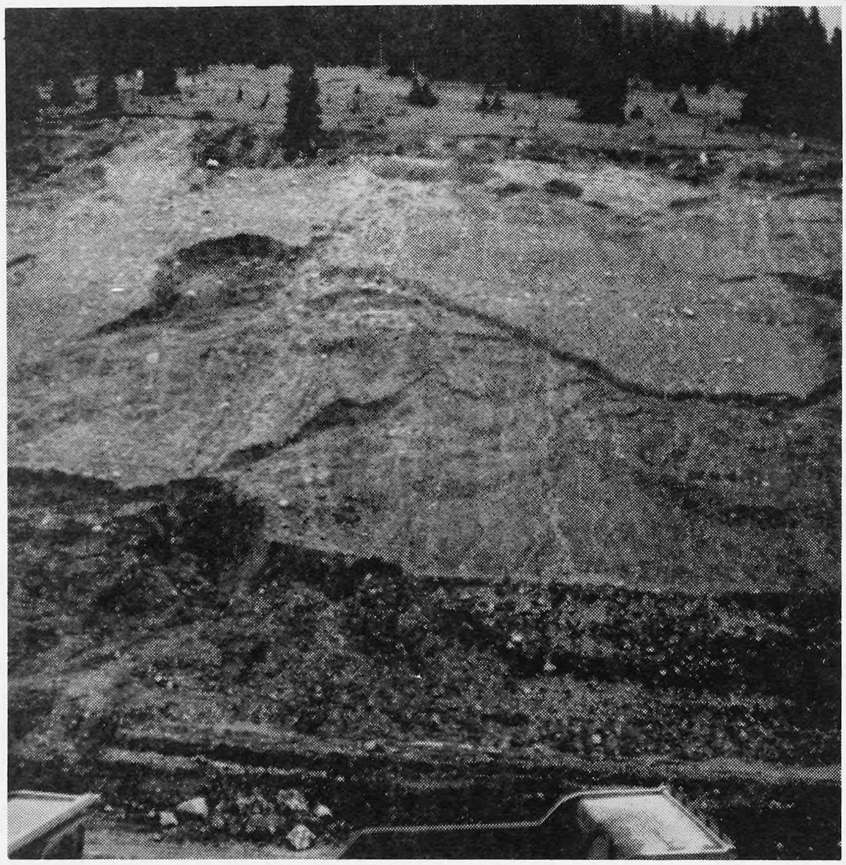


Figure 12. Right cut slope at about Sta. 39+00 in Borrow Area B, September 1976. Note failures indicated by multiple scarps. This picture is typical of conditions in the cut slope between Stas. 36+50 and 41+00.

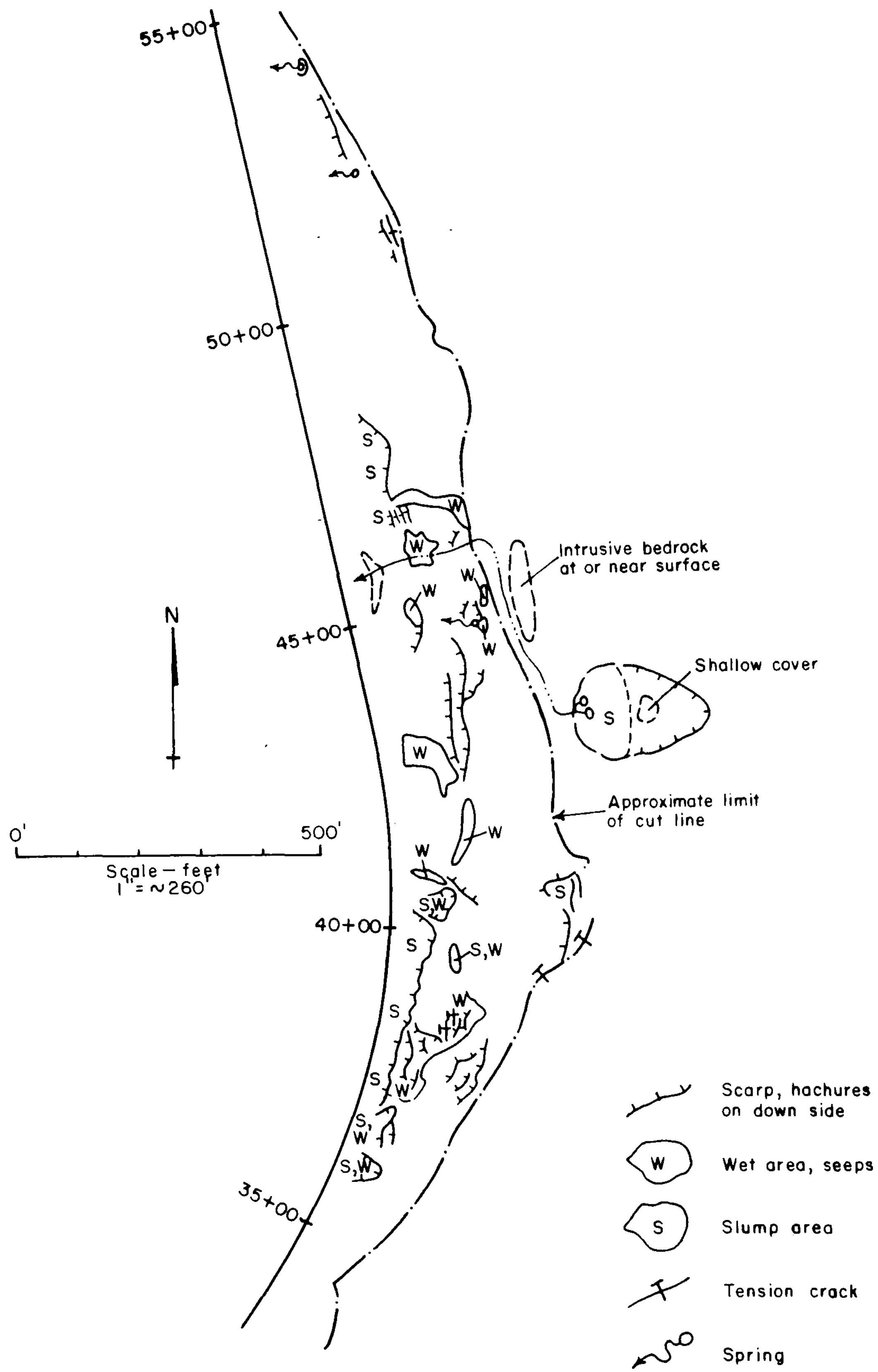


Figure 13. Generalized Map of Borrow Area B early September, 1976.

arcuate breaks in slope which appears to be old landslide features were also noted in this zone. This fault passes under the floodplain of Mayflower Creek where a thick embankment is planned. It is concluded that any minor motion along this fault, however unlikely, would probably be compensated for in adjustments of the fill material.

A third fault crosses the alignment between Stas. 165+00 and 166+50 as shown in Figure 3. A 12-foot scarp, a large boggy, mossy, unstable area with numerous "pistol-butted" trees and a large area of slope wash were observed within and adjacent to the fault zone. This slope wash is the only major breach in the lateral moraine along Tenmile Creek.

3.5.3 Tenderloin Area

An area of extremely unstable surface conditions had been known and studied along the alignment from approximately Sta. 115+00 to Sta. 157+00 (see Fig. 11). Figures 14, 15 and 16 illustrate density of vegetation cover, and unstable morainal materials in this area, which has been nicknamed the "tenderloin." Numerous scarps and related evidences of failure, as well as dozens of seeps, springs and small boggy areas filled with lush, water-loving vegetation indicate the general instability of this reach of the alignment. The surficial deposits in this area appear to be a relatively thin, highly irregular mantle of till and colluvium.

The ground surface in the "tenderloin" is highly disturbed, most likely the result of mass-wasting processes. These processes cause the saturated unconsolidated material lying on oversteepened slopes to move downhill. In an effort to determine the nature and magnitude of this downslope movement the Colorado Department of Highways had installed inclinometer test holes at Stas. 124+30 and 138+50 in December, 1975. Downhill deflection in these holes was monitored bimonthly from the last half of May, 1976 through the first half of September, 1976. The data generated from this program was erratic, and it was decided that no further evaluation should be made after other more reliable information was collected during the field mapping.

Slope failures in this area occurred on surfaces with slopes as shallow as 4:1 and some scarps were measured with as much as 35 feet of relief. More than 50 small-scale failures were mapped from Sta. 115 +00 to 160+00 within the mapped zone. Much of this area had natural slopes of 2:1 that had not failed. This stability is in part due to the cohesive nature of the vegetative mat.

Depth to bedrock was of greater concern in the "tenderloin" than in any other part of the alignment. In order to make an estimate of this depth, geophysical and test pit programs were conducted to supplement information acquired from surface geologic mapping and from a few exploration test holes. Figure 17 shows the results of this effort.

The possibility was considered that some of the excess moisture in the "tenderloin" was derived from No. 1 canal (1000-1500 feet uphill). It was determined, however, from the owner that no water was released through this canal in 1976.

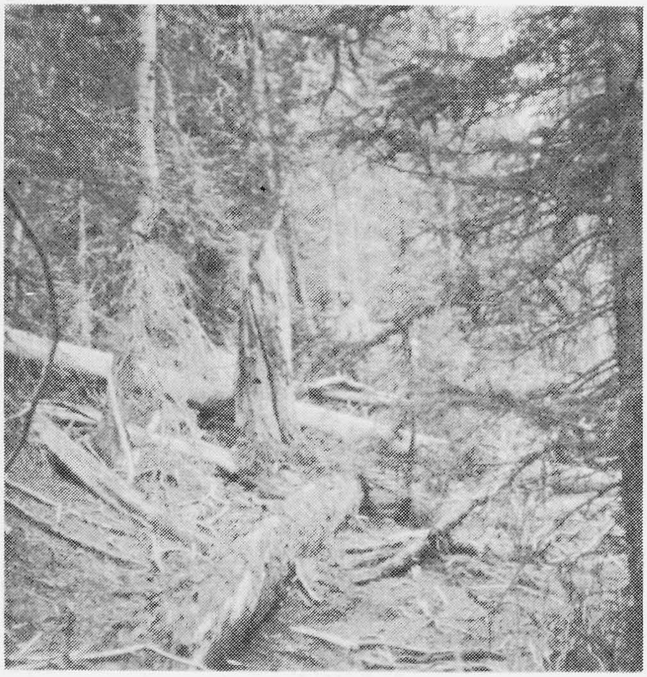


Figure 14. Dense vegetation in "tenderloin" area.

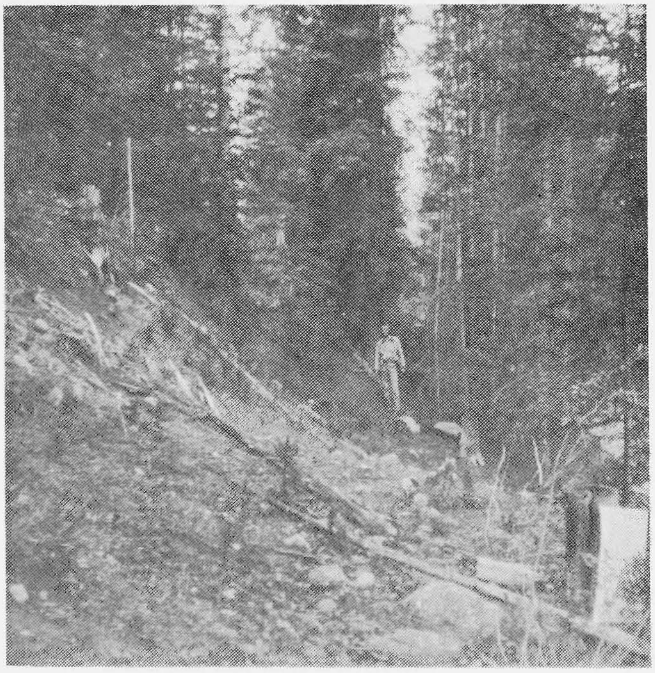


Figure 15. Unstable slope in morainal deposits, "tenderloin" area.

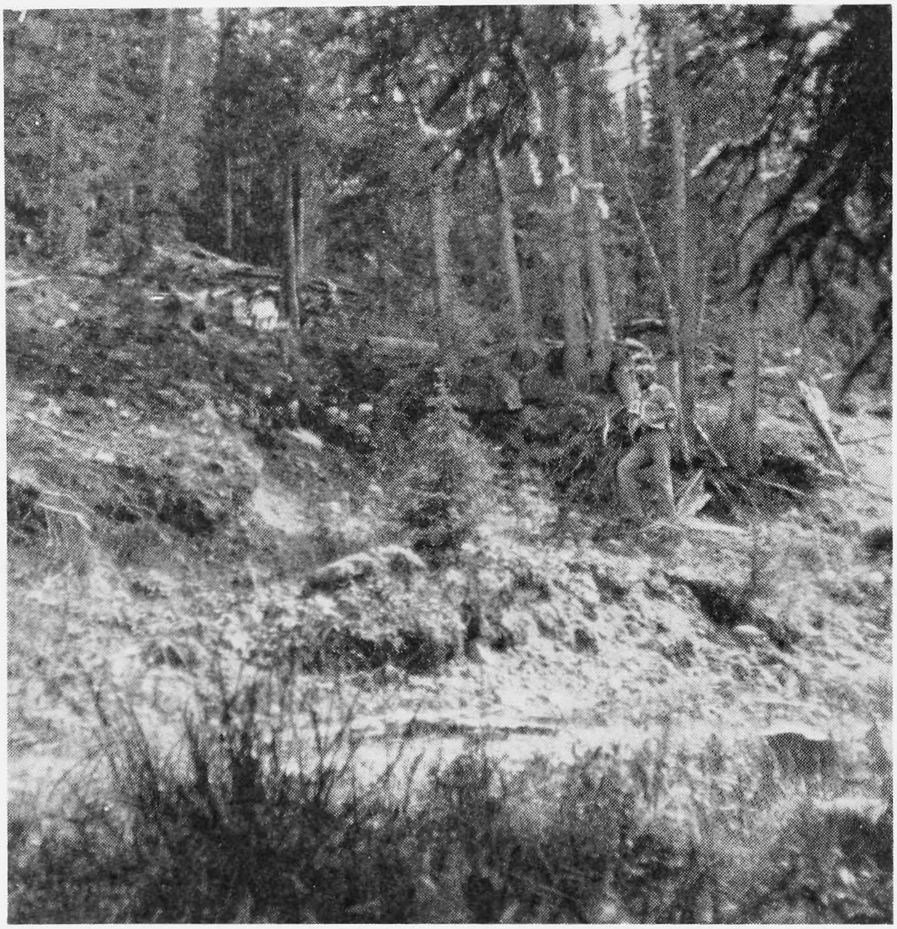


Figure 16. Multiple scarps above an old drill location access road at about Sta. 145+00 in the "tenderloin" area.

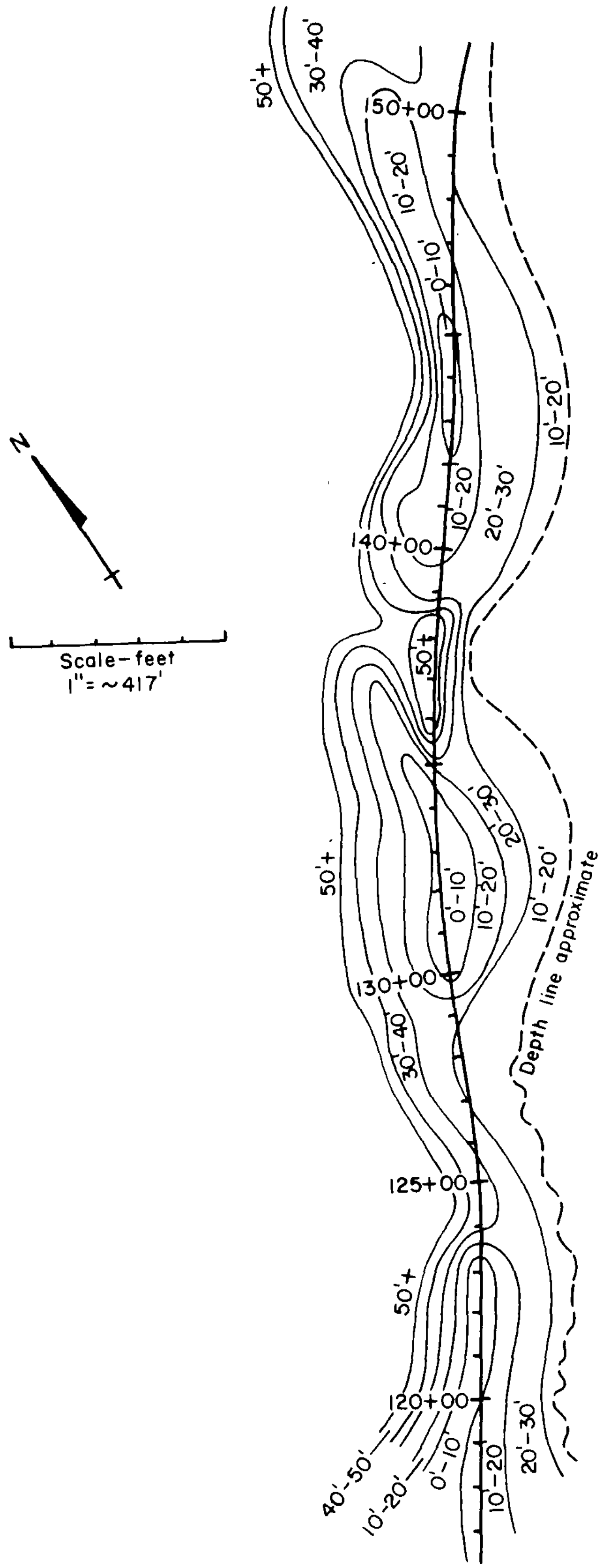


Figure 17. Depth-to-Bedrock Map
Tenderloin Area

3.5.4 Talus Train

A talus train is mapped across the alignment from Sta. 180+00 to 182+50 (Fig. 3). This deposit includes large angular to subangular cobbles and boulders in a highly unstable condition. Many of these boulders are large enough to present a hazard during construction.

3.5.5 Oversteepened Morainal Slopes

Lateral moraines along Tenmile Creek in the reach from Stas. 198+00 to 206+00 are in a naturally oversteepened condition with slopes up to 60° (nearly $\frac{1}{2}$:1) measured during the field mapping. These slopes on thick unconsolidated deposits are identified as a possible construction hazard particularly if undercut when wet. Excavation of these steep morainal slopes is recommended in the late summer when moisture conditions should be most favorable to the stability of the material.

3.5.6 Mine Subsidence

Several old abandoned mine workings are on the new alignment around Sta. 95+00 and Sta. 135+00. These areas are potential sites for ground subsidence and consequent failure of, or damage to, the road surface. It is concluded that deep cover gained within short distances above the old mine workings should minimize the subsidence potential.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 The following types of problems or potential problems exist along the alignment:

a. landslide and slump on both disturbed and undisturbed ground, and in cut and fill,

b. shallow perched water tables causing soft ground with attendant instability problems during and after construction,

c. excessive surface runoff with associated flooding and erosion,

d. subsidence of the roadway due to undermining.

4.2 The following problem areas have been identified:

a. Borrow Area B (Sta. 35+00 to 54+00) - A high cut slope in glacial till has undergone and is undergoing failure due to excess moisture in clay-rich unconsolidated material. Engineered drainage-handling plans will be required in this area.

b. Fault Zones - Three faults or fault zones cross the projected alignment. Absence of surface evidence of faulting implies that these faults are inactive and no problems should be associated with them. Structurally unsound material may be associated with the faults and this material will require stabilization or removal and replacement.

c. "Tenderloin" Area (Sta. 115+00 to 157+00) - This is an area of extremely unstable surface conditions related to excess moisture in unconsolidated material. In this reach over 50 small-scale failure features are mapped some of which occur on slopes as shallow as 4:1 and with as much as 35 feet of relief. The vegetative cover in this reach provides a substantial cohesive element as well as protection from erosion by surface runoff.

d. Talus Train (Sta. 180+00 to 182+50) - This reach includes a talus train comprised of many large cobbles and boulders.

e. Oversteepened Morainal Slopes (Sta. 198+00 to 206+00) - Oversteepened slopes up to 60° (nearly ½:1) exist on unconsolidated till in this reach. These deposits should be excavated in small lifts with relatively small equipment during favorable ground moisture conditions.

f. Mine Subsidence - Old abandoned mine works lie close to the projected alignment in the vicinity of Stas. 95+00 and 135+00. Any potential for ground subsidence in these areas should be corrected before emplacement of embankment.

4.3 Surface mapping and subsurface data indicates that the depth to bedrock varies from the present ground surface to greater than 100 feet in some locations. Test pit and seismic information were reasonably complementary and demonstrated a simple, relatively uniform bedrock surface.

4.4 Drainage is critical to the stability of most of the surficial material on which the highway will be founded. Detailed, site-specific, drainage design will be required in critical locations.

4.5 Till and colluvium cover large portions of the alignment. These deposits should be stable in well drained areas but relatively unstable in the presence of excess moisture. Locally these deposits have failed by slumping or sliding on slopes as shallow as 4:1. Much of this material stands in an oversteepened condition (2:1) by a stabilizing vegetative mat overlying. Disturbance of this mat will reduce slope stability.

4.6 Both competent and incompetent bedrock probably will be encountered in cuts along the alignment. Orientation and spacing of bedding, joint, fracture and foliation planes and the presence or absence of water will determine the degree of safety or hazard in any individual cut.

4.7 The absence of lenses of clay or organic matter in the test hole drilled in Mayflower Creek indicates that the thick alluvial deposit will support the extensive artificial fill after the surface has been cleared.

4.8 Much of the excavated material along sections of cut may be acceptable as embankment fill. Side-casting of unusable zones of clayey material, organic matter and fault gouge will be required.

4.9 Due to the inconsistent and erratic nature of the inclinometer hole data collected by the State Highway Department, we concur with their preliminary appraisal that the data cannot be readily evaluated.

Although these problem areas have been identified, the nature of the geology along this alignment is such that many field decisions will be required during construction. This is expected to be particularly true of the "tenderloin" area.

5.0 SELECTED REFERENCES

The following is a list of references used in the course of the project and the preparation of this paper.

1. American Metal Climax Inc. Geology Staff.
   a. Figure 3 North Kokomo-Surface Geology.
   b. Aerial photographs of the proposed alignment.
   c. Test pit and drill hole information.

2. Bergendahl, Max H., 1963, Geology of the Northern Part of the Tenmile Range, Summit County, Colorado: USGS Bulletin 1162-D.

3. ---1969, Geologic Map and Sections of the Southwest Quarter of the Dillon Quadrangle, Eagle and Summit Counties, Colorado: USGS Misc. Geologic Investigations Map I-563.

4. ---and Koschmann, A.H., 1971, Ore Deposits of the Kokomo-Tenmile District, Colorado: USGS Prof. Paper 652.

5. Deere, D.U., Merritt, A.H. and Coon, R.F., 1969, Engineering Classification of In-situ Rock: AFWL-TR-67-144, Air Force Weapons Laboratory, Kirtland Air Force Base, New Mexico.

6. Division of Highways, State of Colorado. The following unpublished information was made available to us.
   a. Data collected from borehole inclinometer surveys conducted in 1976.
   b. Original and revised profiles along the alignment (dated 8/17/76 and 8/31/76 respectively).
   c. Preliminary soil surveys.
   d. Seismic surveys.

7. Klauber, William (of Climax), 1975 (?), Surficial Geologic Map of the "Tenderloin" Area, ±Sta. 110+00 ± Sta. 146+00; including results of Climax and CHD geophysical surveys, and Climax test pit program (1975).

8. National Oceanic and Atmospheric Administration/Environmental Data Service, 1975, Climatological Data: Colorado Annual Summary 1975, vol. 80, no. 13.

9. Woodward-Clevenger & Associates, Inc., 1973, Engineering and Geologic Investigations, Proposed Clinton Gulch Dam Near Climax, Colorado (prepared for Amax).

10. ---1973, Engineering and Geologic Investigations for Mayflower Tailing Disposal Project Climax Mine Near Climax, Colorado (prepared for Amax).

Epilogue

Construction on the realignment began in the spring of 1977 under the supervision of the Colorado Department of Highways. The highway was placed in service in September, 1978. The limited construction season forced a shutdown from late 1977 until late spring, 1978.

In early 1979, Centennial Engineering, Inc. submitted the design of this highway to the Colorado Consulting Engineers Council in a competition for the best design and construction project, completed in 1978, under $5,000,000. Centennial won this competition and received an award for it in May, 1979.

During the construction period monthly inspections were made by the design engineer, the soils engineer, and the engineering geologist. We had concluded in our report that many field decisions would have to be made, and this prediction proved to be true, particularly in the "tenderloin" area.

The two problem areas of most concern in the initial investigation had been the Borrow Area B cut slopes, and the "tenderloin" area. Inspections concentrated first on Borrow Area B, and later on the "tenderloin." On July 31, 1982, an informal inspection trip was made and a series of pictures was taken along the alignment.

Figure 18 shows Borrow Area B and the stabilizing berm which was emplaced to load the base of the cut. The crest drain (see Fig. 19) along the top of the cut was performing as intended although some material had sloughed down into the permeable drain. Some slumping (see Fig. 20) had occurred in the vicinity of Sta. 45+00 and this appeared to be due to excess water in a clayey environment.

As evidence of the concern with the environment, Climax planted several thousand seedlings in various cuts along the alignment in addition to hydromulching all cuts. Figure 21 shows hydromulching under way during November, 1977, with snow on the ground. Although the evergreen seedlings grow slowly, the hydromulching has been effective as witnessed throughout all of the alignment where bare rock is not exposed.

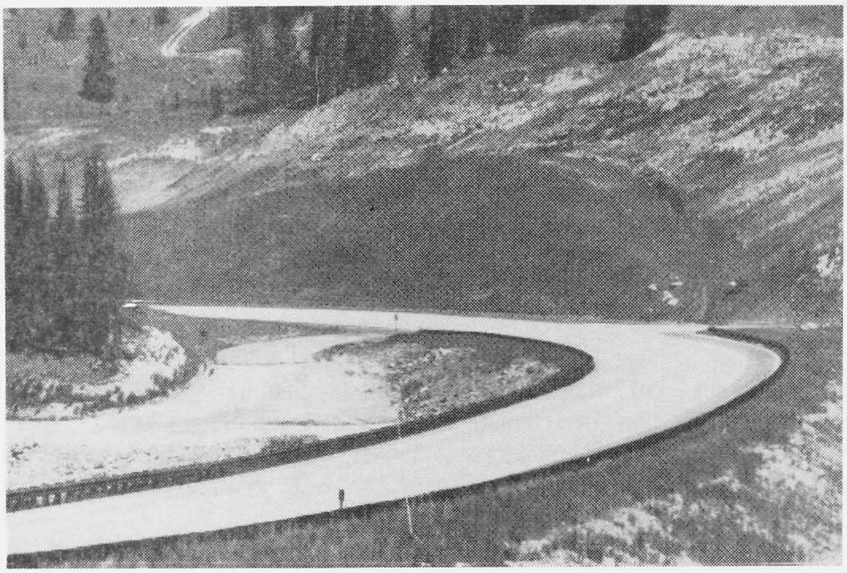


Figure 18. Borrow Area B. Berm (dark arcuate area) in lower part of cut has been an effective barrier to major failure.

Figure 19. Crest drain on right above Borrow Area B. cut. Drain is effectively transmitting water through porous blanket although some sloughing has occurred.

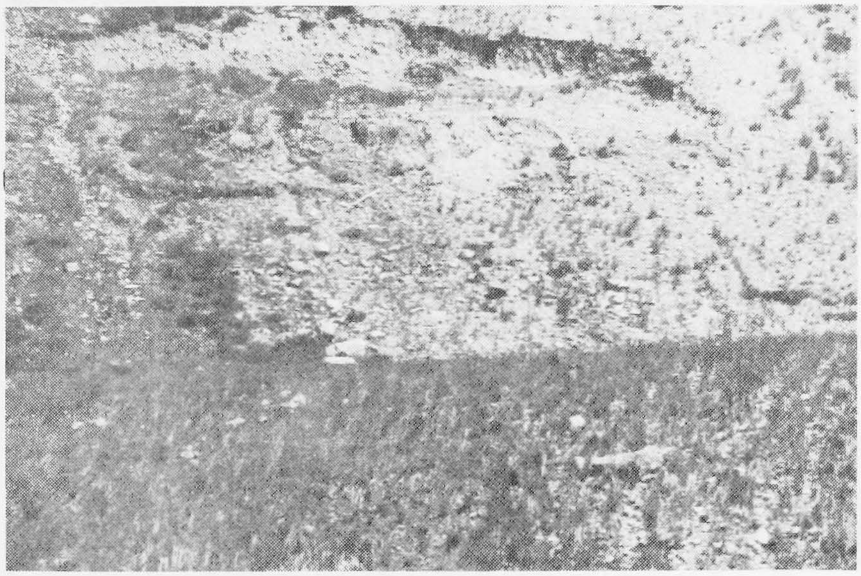


Figure 20. Minor sloughing of morainal material in Borrow Area B above berm surface (lower third of picture).

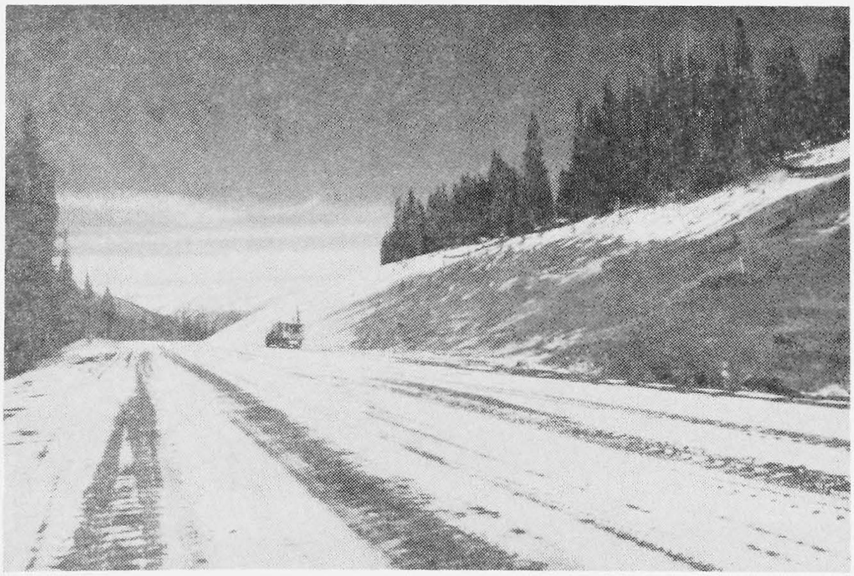


Figure 21. Hydromulching, November 1977.

The system of subdrains placed in the "tenderloin" area was of particular interest. Figure 22 is a panoramic view of the "tenderloin" in which it can be seen that no major failures have occurred. Minor slump features such as the one shown in Fig. 23 are uncommon through this reach of the alignment.

In summary, it is apparent that almost six years after the highway was put in service no failures of consequence are apparent. This alignment is therefore a good example of what intensive engineering geologic analysis, soils engineering, and design engineering can accomplish with responsive construction supervision.

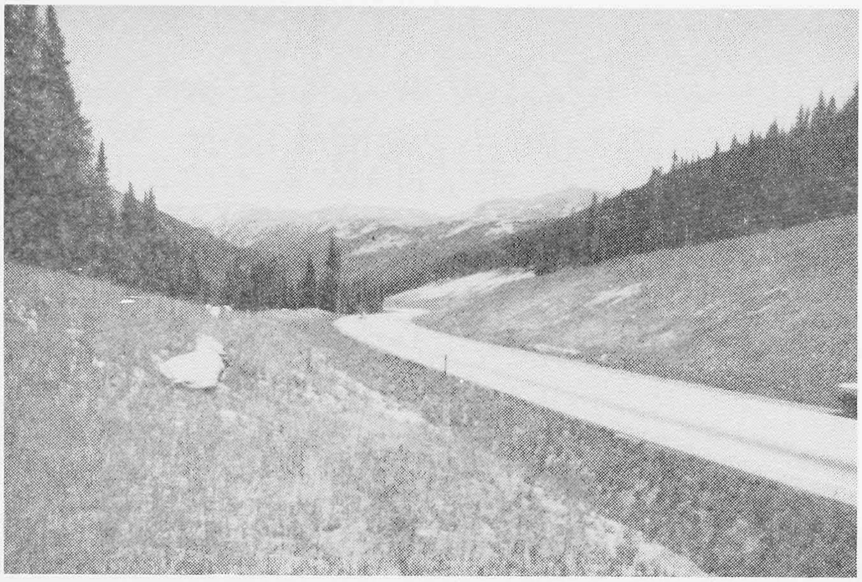


Figure 22. Panoramic view of "tenderloin" area. Note the well established stand of grass in the foreground. Rocks such as the large boulder to the left were placed at random along some of the slopes as a matter of aesthetics.

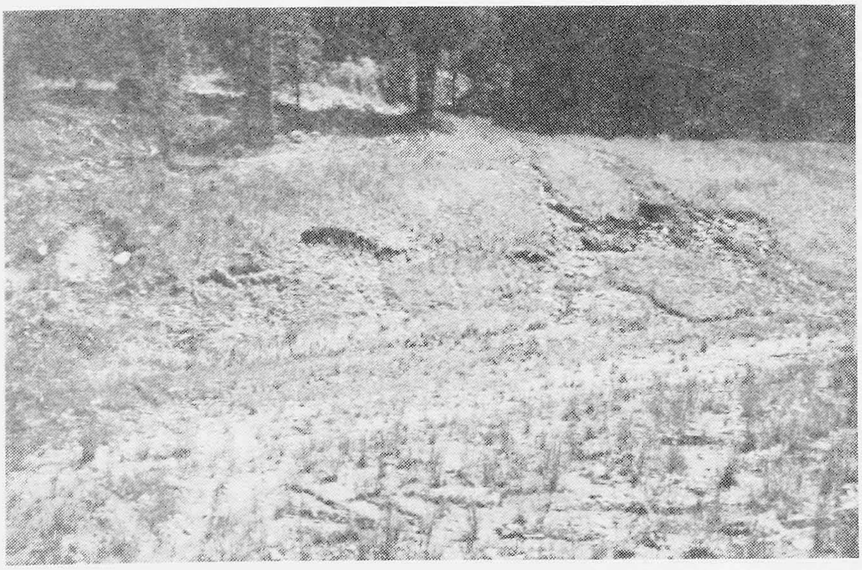


Figure 23. Minor slump features in the "tenderloin" area just south of Humbug Creek.

SLOPE STABILITY CONSIDERATION OF THE COLORADO STATE HIGHWAY 91 RELOCATION

By Darrel V. Holmquist
CTL/Thompson, Inc.
1971 West 12th Avenue
Denver, Colorado 80204

ABSTRACT

The relocation of Colorado State Highway 91 presented some unique slope stability problems and solutions. In Borrow Area B, a 155-foot tall cut slope experienced multiple failures caused by oversteepened slopes and ground water. The natural topography precluded flattening the slope adequately to achieve desired stability. Surface and subsurface drainage, slope flattening and buttressing were used to achieve the desired stability. The area known as the Tenderloin was approximately one mile in length and contained over a hundred natural landslides, springs, seeps and boggy areas in steep, heavily forested topography. Comparative analyses indicated cut sections with subsurface drainage should provide the highest long-term stability, even though minor failures could occur during construction. The highway was completed in 1978 and has been performing satisfactorily since its opening.

INTRODUCTION

The relocation of State Highway 91 was necessitated by the expansion of the Mayflower Tailing Pond by the Climax Molybdenum Mine. The realignment was roughly 22,500 feet in length at elevations ranging from approximately 10,200 to over 11,200 feet. A survey of the alignment indicated two potential problem areas which were commonly known as Borrow Area B and the Tenderloin (Fig. 1). Borrow Area B was a cut approximately 155 feet in height and 2,000 feet in length. The excavation was performed to provide borrow material for construction of the adjacent dam across Clinton Gulch. The Tenderloin was a natural area, approximately 5,700 feet in length and was bordered to the south by Mayflower Creek and to the north by Humbug Creek.

Both Borrow Area B and the Tenderloin exhibited highly unstable conditions. These areas were investigated by geologic and geotechnical investigations designed to determine the causes of the instability and provide economical alternatives to reduce the probability of failures occurring after the roadway was completed. The geology of the alignment is discussed in a companion paper by Ivey and Hansen entitled "Engineering Geology, Relocation of State Highway 91 Climax Mine Area, Summit County, Colorado". This paper describes the geotechnical investigations, analytical methods used and results, recommended design and construction details and performance history of the project.

BORROW AREA B

The Borrow Area B cut exposed glacial till generally classified as a clayey gravel containing substantial cobble and boulder sized particles. The cut slope on the west side of the alignment was not significant in height and appeared relatively stable. On the eastern side where the excavation was as much as 155 feet in depth, numerous slope failures and springs or seeps were noted, the cut was sloped at approximately 1.75:1 (horizontal to vertical) (Fig. 2). The natural slope above the cut ranged in steepness from 2.5:1 to 3:1 and appeared to be relatively stable with the exception of an area where a series of springs emerged from an adit.

Based upon site observations, the area was classified as having three stability conditions. The zone between Sta. 36+00 and Sta. 41+00 has numerous large, shallow seated, slope failures and is classified as a failed zone. Between Sta. 41+00 and Sta. 48+00 there are relatively few minor slumps, but moist to wet areas on the surface indicate that this area is unstable and on the verge of failure. The zone extending from Sta. 48+00 through Sta. 64+00 is relatively stable and shows little water and only a few minor sloughs and was believed to be the most stable of three areas.

The investigation consisted of site observations, field in-place density tests, drilling of three exploratory borings, and laboratory stength testing. The logs of the exploratory borings indicate relatively uniform glacial till material extending to depths of more than 35 feet below existing grades. Two of the borings were drilled along the alignment with the third at the top of the cut slope. Samples obtained from excavations and exploratory borings were used to prepare compacted samples and tested in direct shear to obtain strength values. Observations as well as the check of ground water levels in the exploratory borings suggested a series of perched water tables were present which emerge from the cut along lenses of relatively impermeable clay.

TENDERLOIN

The Tenderloin was a rugged, heavily forested area of steep natural slopes and erratic subsurface conditions. Upslope from the proposed alignment vegetation is less dense and the topography more uniform. The geologic survey performed by Amuedo and Ivey indicated over 100 natural landslides and springs as well as areas with unusually high moisture contents and dense willows (Figs. 3 and 4).

Because of the rugged topography, there was no access for drill rigs and the field investigation consisted primarily of excavating test pits using a large Caterpillar 225 backhoe. The subsurface conditions encountered in the exploratory test pits were highly erratic throughout the Tenderloin area. Conditions ranged from +25 feet of silty, glacial till to fractured quartz monzonite at the surface. An evaluation of the data suggested the critical section consisted of 0.5 to 1.5 feet of forest topsoil overlying highly variable colluvium, slope debris and weathered bedrock. A soft, weathered intrusive was encountered above the bedrock at many locations. Bedrock topographic information was obtained from the Colorado Department of Highways geophysics data and indicates depth to bedrock varying between 0 and 50 feet along the portions of the alignment investigated. Direct shear testing was conducted on several samples of the materials encountered including the colluvium, slope debris and the weathered intrusive. The bedrock encountered was generally a quartzite which had been highly fractured. Fracture spacings as close as 2 to 4 inches were encountered. The quartzite bedrock was found in small pieces, however, its mass was usually ripped using the backhoe. Other zones of bedrock encountered included weathered granite and monzonite.

ANALYTICAL METHODS AND RESULTS

Based upon the field conditions observed, topographic information, laboratory data, and assumed ground water elevations, typical profiles for several configurations and strength conditions were developed. Existing conditions where failures were observed for both Borrow Area B and the Tenderloin were analyzed as a back-check of laboratory strength values. Additional profiles were developed for alternative alignment templates, drainage conditions and strength assumptions to provide a basis for development of design and construction recommendations.

The analytical methods used included the graphical "Swedish Slip Circle Method" and the "Spencer's Technique" as modified by Stephen G. Wright of the University of Texas. Both are method of slices-limit equilibrium techniques, where the "factor of safety" is defined in terms of the ratio of resisting forces to driving forces and a value of 1.0 indicates incipient failure. The Swedish method is considered an approximate analysis technique, since only moment equilibrium is considered. Spencer's method satisfies complete mass and individual slice, force and moment equilibrium and is considered an accurate technique. The primary advantages of the Swedish method are that it is easily and quickly performed using hand calculations. Spencer's method requires the use of a computer to efficiently handle the mass of calculations required. The computer code used was Stephen Wright's "SSTAB1". Typically, the Swedish method was used for parametric analyses and for back-calculating strength values from failed slopes. The Spencer's method was used for critical analysis conditions and where a large number of failure circules were to be evaluated.

An iterative analysis procedure was used to develop recommended design and construction details. The existing failures offered a full scale check of the analytical model. Back-calculations of required strength were used to provide some confidence in the laboratory developed strengths. The calculations at Sta. 39+00 (Fig. 5) indicated a minimum required strength of 400 psf cohesion and an angle of internal friction of 25°. This compared favorably with laboratory values of 25° and 800 psf. Erratic conditions in the Tenderloin limited the usefullness of this technique and parametric analyses were used to determine the most critical parameter affecting stability. The results indicated the water table location can decrease the factor of safety as much as 50 percent, while a 50 percent increase in strength results in only a 25 percent change in the factor of safety.

Comparisons were made to determine the effect of template alternatives in the Tenderloin. Using similar topographic conditions and identical strength values, cut and fill alternatives were analyzed. Analyses indicate providing cut-type templates with the resulting lower water table yielded the most stable conditions. In some cases, it was difficult to obtain even a marginally safe slope for a fill configuration without fully draining the slope.

As shown on Fig. 7, alternative slopes ratios were analyzed for Borrow Area B and suggested that slopes steeper than 3:1 (horizontal to vertical) could experience distress and fail. The analyses were conducted for the two strength conditions obtained from laboratory testing and assumed the presence of a relatively high ground water table. Using a perched water table stability was marginal.

The results of the analytical work were used to define potential problem areas, identify possible solutions and provide a rational basis for recommended design and construction details. The conclusions derived included:

1. Strength values obtained from samples designed to model field conditions were consistent with back-calculated strengths for failure observed in the field.

2. Parametric analyses indicated ground water and perched water tables, in both Borrow Area B and the Tenderloin, were primary stability controlling factors in the analyses. Strength of the unconsolidated colluvium and slope debris also affected the calculated factor of safety.

3. The cut alternative template in the Tenderloin was significantly more stable than fill alternatives. The primary reasons for the difference were lowering the ground water and reducing the surcharge load placed on the slope.

4. The slope ratio for Borrow Area B would have to be on the order of 3:1 to provide marginally stable slopes.

STABILITY ENHANCEMENT CONSIDERATIONS

There are only four methods of enhancing the stability of any cut or fill slope. These include: (1) artificially increasing the strength of the soils, (2) flattening the slope to reduce driving forces, (3) buttressing the toe of the slope to increase the resisting forces, and (4) lowering the water table by drainage to increase effective stresses. Enhancement techniques were evaluated in general, then specifically for Borrow Area B and the Tenderloin to determine the most applicable and economical. Artificially increasing strength usually involves removal and replacement, dynamic compaction or electro-osmosis and was considered an uneconomical approach for either area. Flattening slopes in either Borrow Area B or the Tenderloin beyond 2:1 (horizontal to vertical) was not attractive considering the steepness of the natural topography. Buttressing cut and fill slopes was considered appropriate since excess rock was generated from the cut across Clinton Gulch. The most promising alternative was use of surface and subsurface drainage to stabilize the slopes.

Borrow Area B cut slope stability was marginal as shown by performance and analysis. Even use of slope flattening from 1.75:1 to 2:1 did not provide adequate stability unless completely drained. Because the water table was perched upon discontinuous clay lenses, it was not possible to completely drain the slope. The most practical solutions appeared to be utilizing excess rock from a cut located across Clinton Gulch to buttress the fill and provide a series of surface and subsurface drains to reduce the pore water pressures thus reducing the driving forces and increasing the overall strength.

Analytical results shows stability in the Tenderloin was most sensitive to ground water levels. Cut sections were more stable than fill sections mainly due to the drawdown in ground water caused by cut and unloading the head of potential failure surfaces. Although it was desirable to flatten slopes significantly, topographic conditions limited slopes to 2:1 in order to catch existing slopes . These recommendations were believed to significantly reduce the stability during construction and result in some failures. After drainage had reduced the ground water level, the driving forces would decrease resulting in increasing stability with time. Recommendations provided for installation of temporary drains at the beginning of construction to reduce short-term failures and an extensive permanent underdrain system. Over 15,000 lineal feet of subdrain was placed in the 5,700 feet of the Tenderloin alignment. The drain was designed to intercept and conduct water away from cut slope and sensitive fill areas. Fills were benched into the natural soils and bedrock and provided with blanket drains. During construction, changes in topography, ground water, and subsurface conditions required field changes in the design and location of the subdrain system.

The design of subdrains for both areas was based upon use of either a graded aggregate filter or a non-woven geotextile filter system. The alternative designs are illustrated on Fig. 8. Because of the different material types, the aggregate filter system required two specified gradations. One was for the glacial till which contained significant clay and silt sized particles, while the other was developed for the granular colluvium. A non-woven,polypropolyene geotextile was found suitable for either soils condition, less expensive and easier to install.

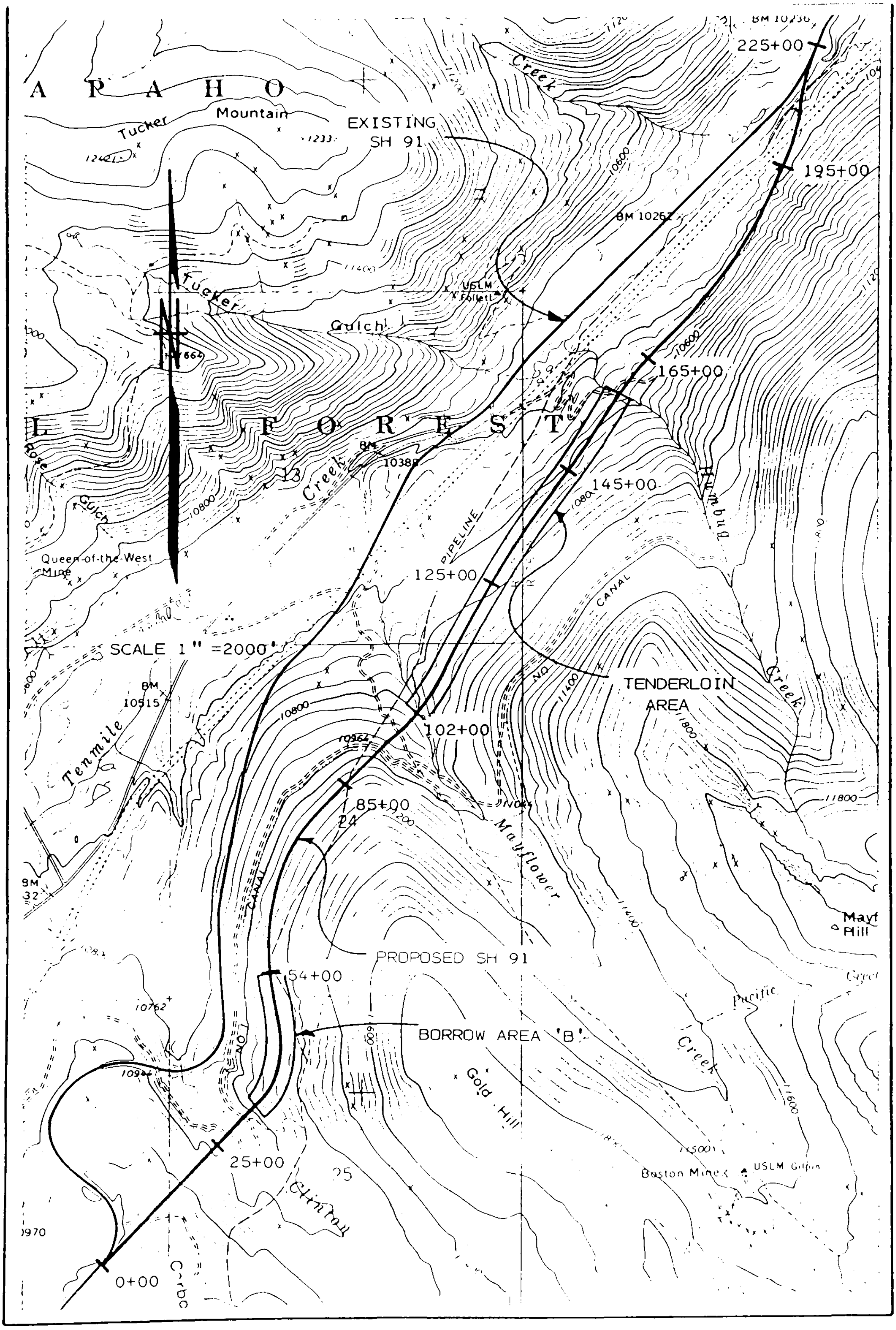
SUMMARY

The construction of State Highway 91 began in May of 1977 with the roadway being opened to traffic in September 1978. The total cost of construction, based upon the engineered design was 4.7 million dollars.

The project involved approximately 1,400,000 cubic yards of cut and 1,150,000 cubic yards of fill with numerous cuts and fills over 100 feet in height. With very few exceptions, the project has performed satisfactorily. An occasional minor slump or slough was detected, but no major movements have occurred even during the wet winter of 1979-1980.

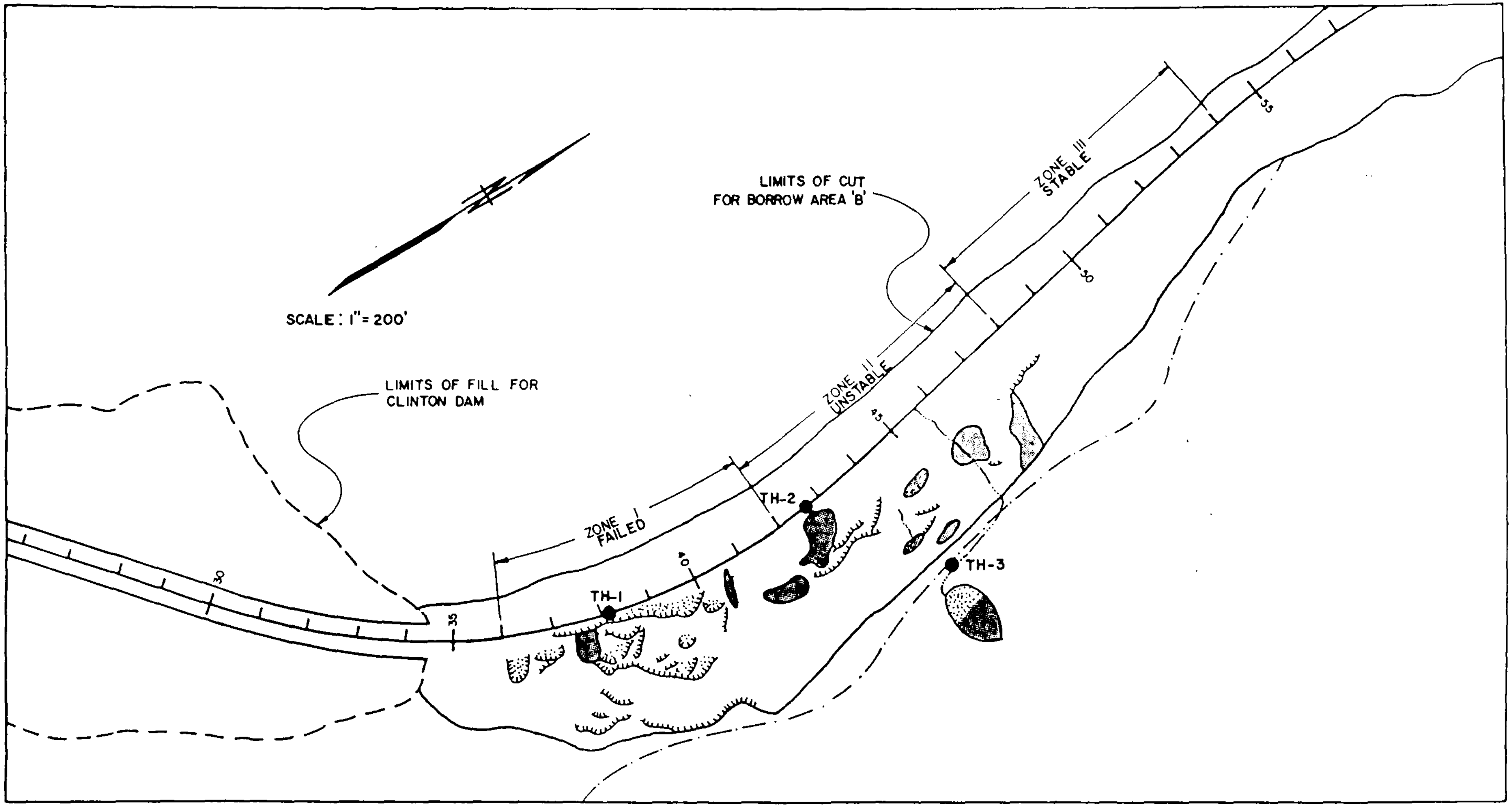
ACKNOWLEDGMENTS

The author wishes to thank the personnel of The colorado Department of Highways and the Climax Molybdenum Mine, particularly Mr. Warren Alloway. A special acknowledgment is also provide to Messrs. John Ivey of Amuedo and Ivey who directed the geologic investigation, Dick Sparlin and Al Menhennett of Centennial Engineering, Inc. who were the project director and advisor. Mr. Bud Noble of Noble Engineering provided review and guidance which was appreciated.



VICINITY MAP

FIG. 1



LEGEND

VERY WET SOIL AND/OR SLOPE DEBRIS. (FAILED)

MOIST TO VERY MOIST SOIL. (NOT FAILED)

SCARP OF SLOPE FAILURE.

FLOWING STREAM.

ABANDONED GASLINE.

TEST HOLE LOCATION.

NOTES:

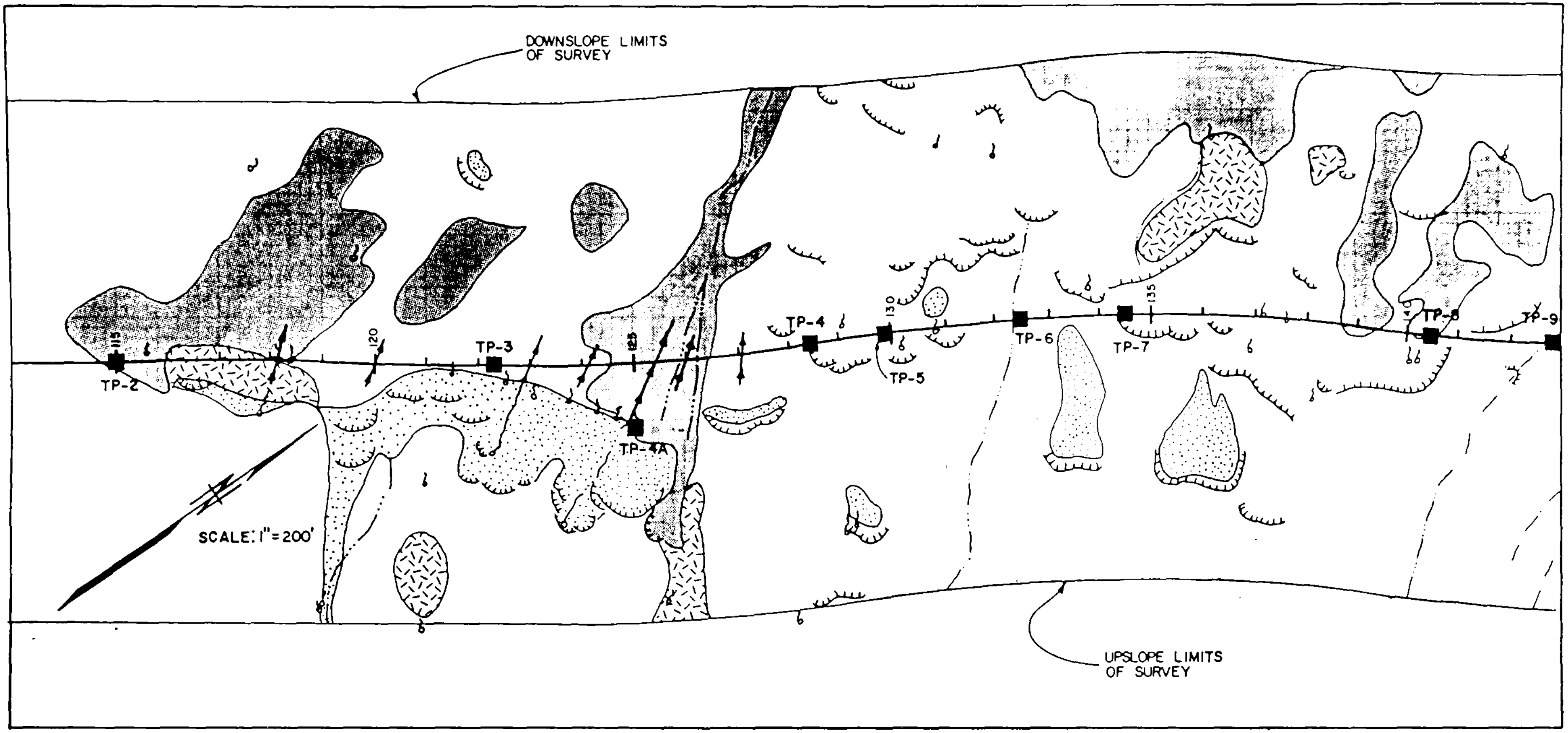
1) LOGS OF SOILS ENCOUNTERED IN TEST HOLES ARE SHOWN ON FIG. 5.

2) FOR MORE DETAILED INFORMATION ON SURFACE GEOLOGY SEE THE COMPANION REPORT PREPARED BY AMUEDO AND IVEY.

BORROW AREA 'B' — SURFACE FEATURES AND TEST HOLE LOCATIONS

FIG. 2

FIG. 3



LEGEND

AREAS OF HIGH MOISTURE USUALLY BOGGY WITH DENSE WILLOWS.

DISTURBED GROUND SURFACE USUALLY VERY MOIST, MAY BE RESULT OF SLOPE FAILURE.

DISTURBED GROUND SURFACE SHOWING SIGNS OF SOIL CREEP, NO DISTINCT SLOPE FAILURES.

SCARP OF SLOPE FAILURE.

FLOWING SPRING OR STANDING WATER.

FLOWING STREAM.

OPEN DRAINAGE TRENCH, EXCAVATED NOVEMBER, 1976.

LOCATION OF TEST PITS EXCAVATED SEPTEMBER, 1976.

NOTES:

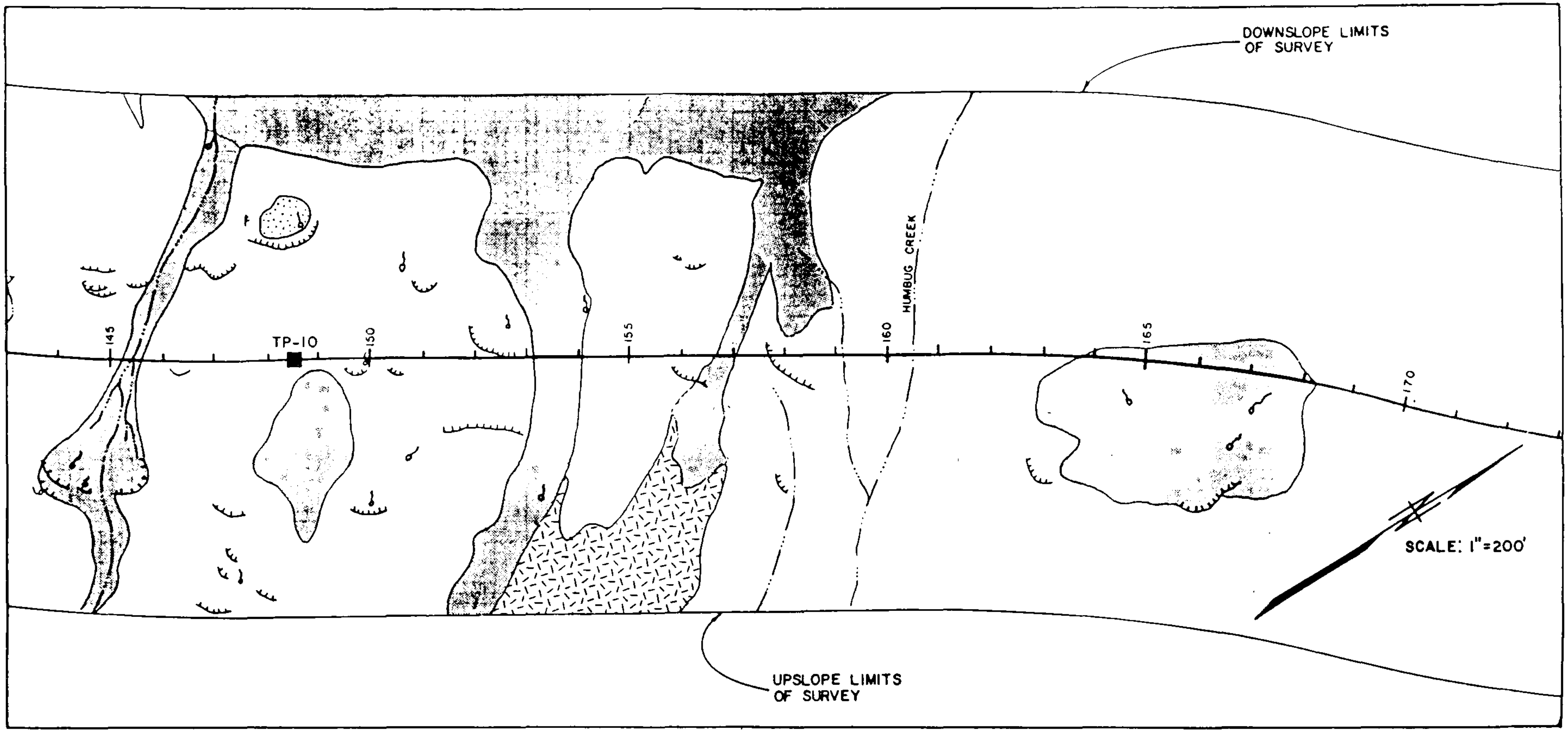
1) TEST PIT LOGS ARE SHOWN ON FIGS. 6 THROUGH 8.

2) FOR MORE DETAILED INFORMATION ON SURFACE GEOLOGY SEE THE COMPANION REPORT PREPARED BY AMUEDO AND IVEY.

3) NO ACCURATE TOPOGRAPHIC INFORMATION WAS AVAILABE AT THE TIME THIS REPORT WAS PREPARED.

TENDERLOIN AREA (STATIONS 115 THROUGH 143) SURFACE FEATURES AND TEST PIT LOCATIONS

FIG. 4



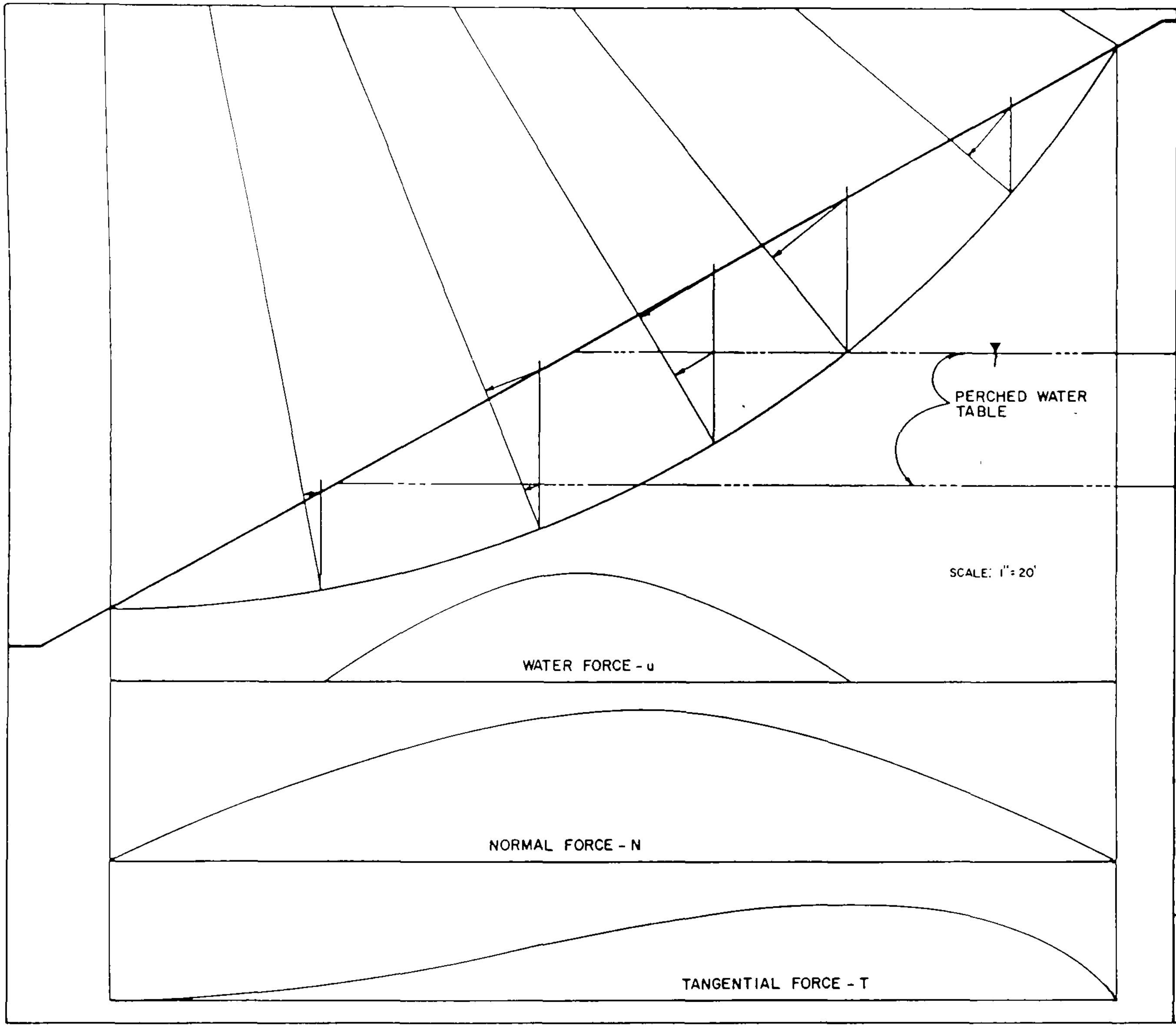
LEGEND

- AREAS OF HIGH MOISTURE USUALLY BOGGY WITH DENSE WILLOWS.
- DISTURBED GROUND SURFACE USUALLY VERY MOIST, MAY BE RESULT OF SLOPE FAILURE.
- DISTURBED GROUND SURFACE SHOWING SIGNS OF SOIL CREEP, NO DISTINCT SLOPE FAILURES.
- SCARP OF SLOPE FAILURE.
- FLOWING SPRING OR STANDING WATER.
- FLOWING STREAM.
- LOCATION OF TEST PITS EXCAVATED SEPTEMBER, 1976.

NOTES:

1) TEST PIT LOGS ARE SHOWN ON FIGS. 6 THROUGH 8.
2) FOR MORE DETAILED INFORMATION ON SURFACE GEOLOGY SEE THE COMPANION REPORT PREPARED BY AMUEDO AND IVEY.
3) NO ACCURATE TOPOGRAPHIC INFORMATION WAS AVAILABLE AT THE TIME THIS REPORT WAS PREPARED.

TENDERLOIN AREA (STATIONS 143 THROUGH 160+) SURFACE FEATURES AND TEST PIT LOCATIONS



NOTES:

1) TOPOGRAPHIC INFORMATION FROM COLORADO DEPT. OF HIGHWAYS (EXISTING CONDITIONS)

2) LOCATION OF PERCHED WATER TABLE IS ASSUMED.

3) STRENGTH ASSUMPTIONS ARE FROM LABORATORY DIRECT SHEAR TESTS.

STRENGTH ASSUMPTIONS:

CASE I	$\phi = 25°$	c = 800 psf	γ = 140 pcf
CASE II	$\phi = 8.5°$	c = 600 psf	γ = 140 pcf

CALCULATIONS:

FACTOR OF SAFETY = F.S.

$$F.S. = \frac{\sum cL + \sum (N-u) \tan \phi}{\sum T}$$

WHERE: c = cohesion
L = length of arc
ϕ = angle of internal friction

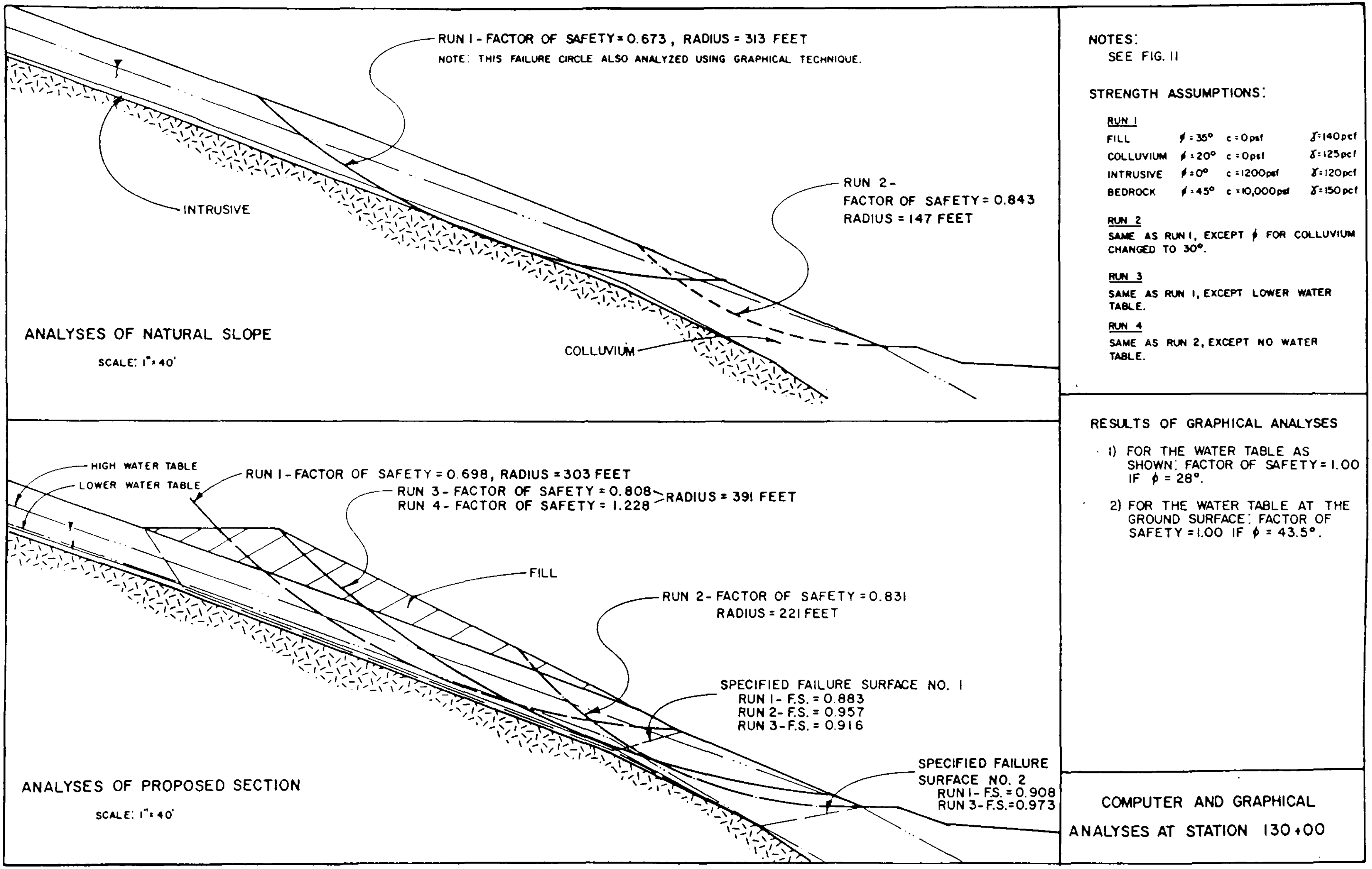
SUMMARY OF RESULTS

FOR FACTOR OF SAFETY = 1.00
$\phi° = 25°$ $c° = 400$ psf

CONDITION	F.S. CASE I	F.S. CASE II
NO WATER TABLE	1.558	0.796
PERCHED WATER TABLE	1.390	0.742

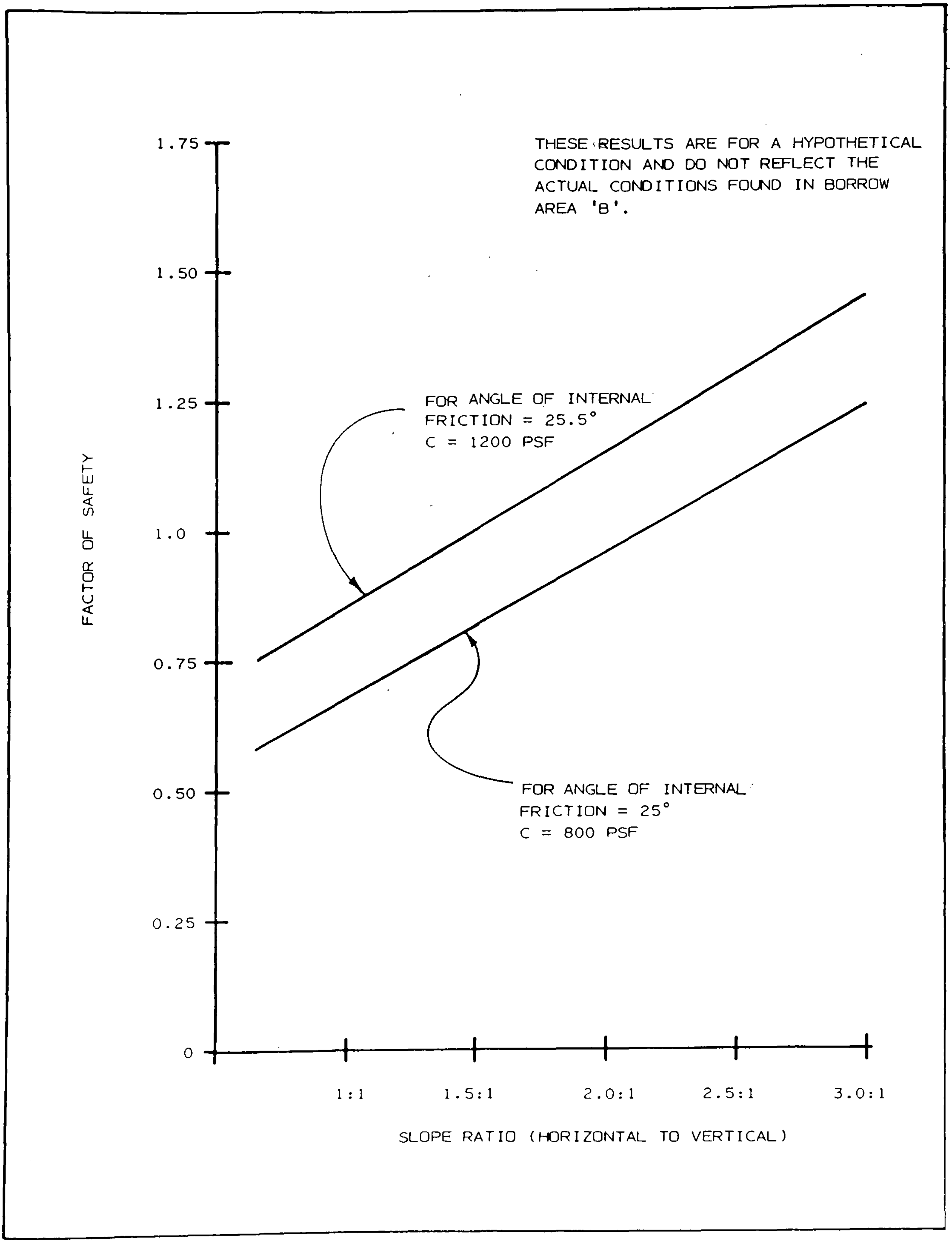
GRAPHICAL ANALYSES FOR STATION 39+00
BORROW AREA 'B'

FIG. 5



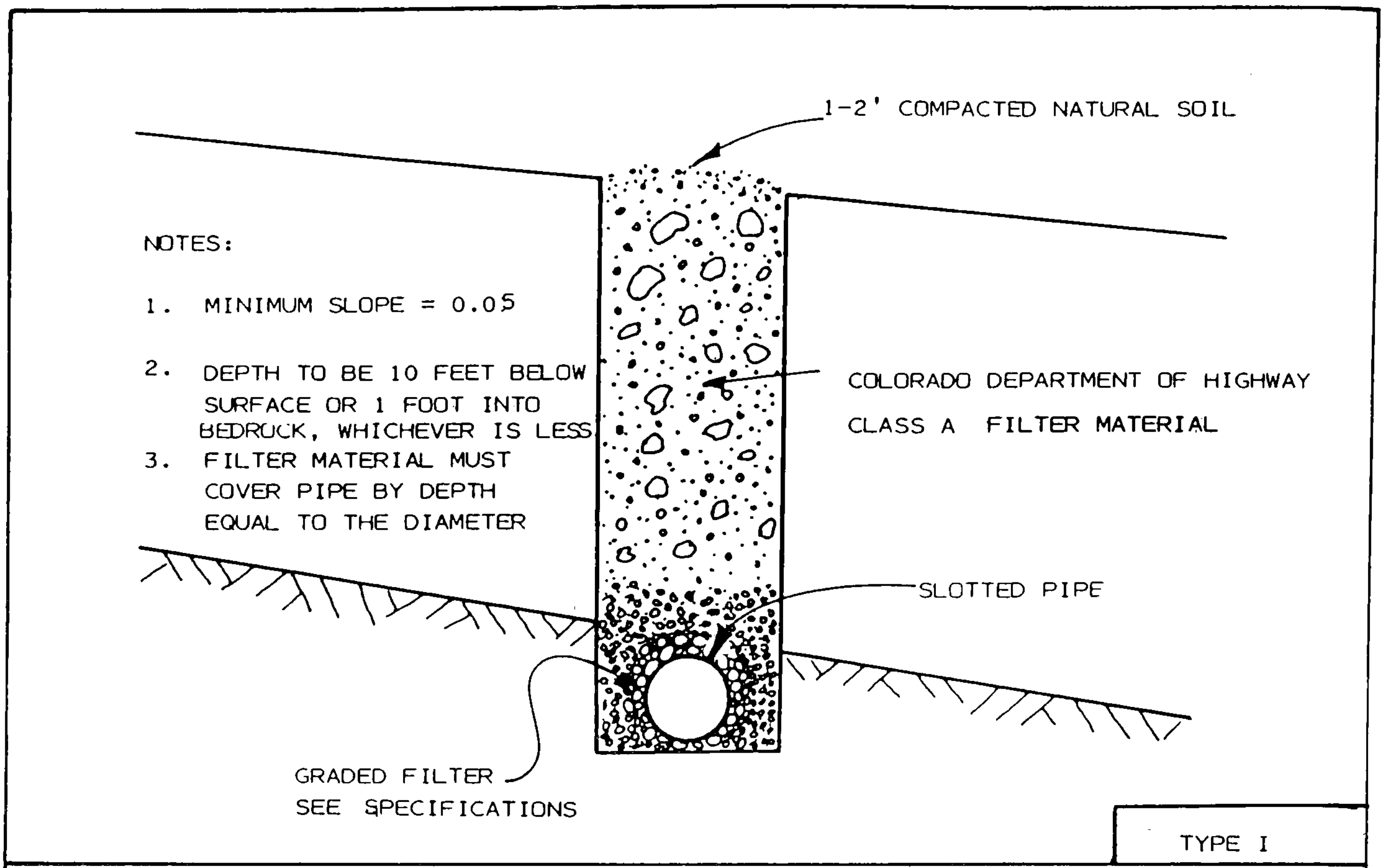
RUN I - FACTOR OF SAFETY = 0.673 , RADIUS = 313 FEET
NOTE: THIS FAILURE CIRCLE ALSO ANALYZED USING GRAPHICAL TECHNIQUE.
INTRUSIVE
RUN 2 - FACTOR OF SAFETY = 0.843 RADIUS = 147 FEET
ANALYSES OF NATURAL SLOPE
SCALE: 1" = 40'
COLLUVIUM
NOTES:
SEE FIG. 11
STRENGTH ASSUMPTIONS:
RUN 1
FILL ϕ = 35° c = 0 psf γ = 140 pcf
COLLUVIUM ϕ = 20° c = 0 psf γ = 125 pcf
INTRUSIVE ϕ = 0° c = 1200 psf γ = 120 pcf
BEDROCK ϕ = 45° c = 10,000 psf γ = 150 pcf
RUN 2
SAME AS RUN 1, EXCEPT ϕ FOR COLLUVIUM CHANGED TO 30°.
RUN 3
SAME AS RUN 1, EXCEPT LOWER WATER TABLE.
RUN 4
SAME AS RUN 2, EXCEPT NO WATER TABLE.
RESULTS OF GRAPHICAL ANALYSES
1) FOR THE WATER TABLE AS SHOWN: FACTOR OF SAFETY = 1.00 IF ϕ = 28°.
2) FOR THE WATER TABLE AT THE GROUND SURFACE: FACTOR OF SAFETY = 1.00 IF ϕ = 43.5°.
HIGH WATER TABLE
LOWER WATER TABLE
RUN 1 - FACTOR OF SAFETY = 0.698, RADIUS = 303 FEET
RUN 3 - FACTOR OF SAFETY = 0.808
RUN 4 - FACTOR OF SAFETY = 1.228
RADIUS = 391 FEET
FILL
RUN 2 - FACTOR OF SAFETY = 0.831 RADIUS = 221 FEET
SPECIFIED FAILURE SURFACE NO. 1
RUN 1 - F.S. = 0.883
RUN 2 - F.S. = 0.957
RUN 3 - F.S. = 0.916
SPECIFIED FAILURE SURFACE NO. 2
RUN 1 - F.S. = 0.908
RUN 3 - F.S. = 0.973
ANALYSES OF PROPOSED SECTION
SCALE: 1" = 40'
COMPUTER AND GRAPHICAL ANALYSES AT STATION 130+00

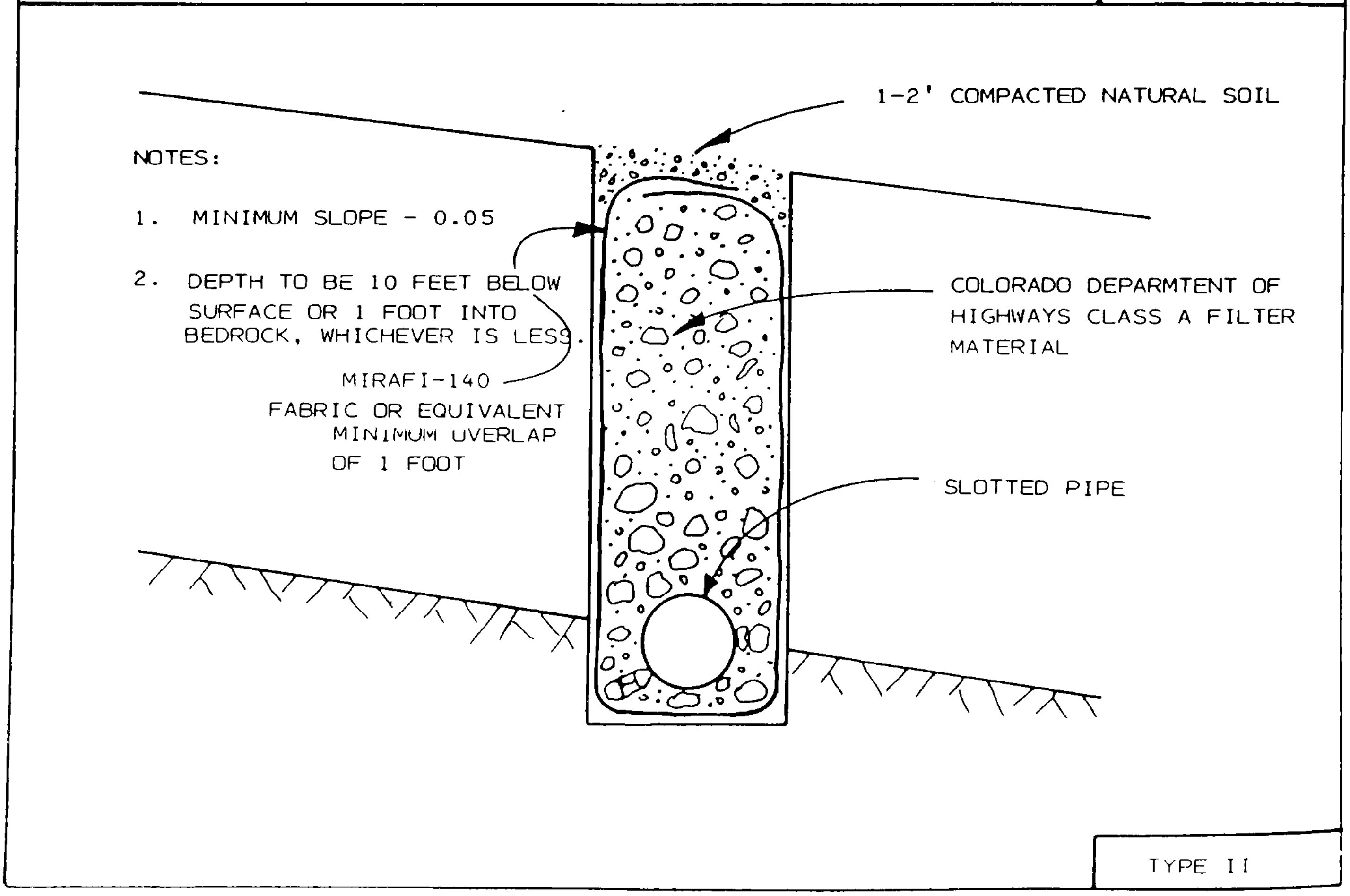
FIG. 6



SLOPE CHART (BORROW AREA 'B')

FIG. 7





TYPICAL DRAIN DETAILS

FIG. 8

APPLICATION OF VACUUM HORIZONTAL DRAINAGE

by

R. Pakalnis
Steffen Robertson and Kirsten

and

T. Lutman
Aardvark Drilling Sales

APPLICATION OF VACUUM HORIZONTAL DRAINAGE

Pakalnis, R.
Steffen Robertson and Kirsten
Vancouver, B.C.
Canada

and

Lutman, T.
Aardvark Drilling Sales, Ltd.
Langley, B.C.
Canada

A modern advance in slope depressurization is the use of vacuum augmented horizontal drains to stabilize slopes. The benefit arises from an increased hydraulic gradient due to the drain exhausting to a negative atmospheric pressure. This paper describes three case studies where the technique has been successfully put to practice. The first is a recent application on a soil slope whereby water flow increased from 0.6 l/min to 9 l/min upon the initiation of vacuum. The second is on a rock slope whereby flows increased from 0 l/min under gravity drainage to 12 l/min upon vacuum initiation. This was associated with a water table drop of 1.1 m. The third case study involved a laboratory model whereby the effects of vacuum drainage were evaluated.

APPLICATION OF VACUUM HORIZONTAL DRAINAGE

INTRODUCTION

Groundwater has a detrimental effect on slope stability in terms of increased hydraulic pressures. Figure 1 depicts a rock slope with a failure plane bounded by a tension crack. It is shown by the above figure that water pressure reduces the shear strength along the failure plane by producing an uplift force. In addition, water within a tension crack creates horizontal hydrostatic pressures which contribute to instability. Brawner (1) suggests that the frictional resistance would decrease by 37 percent due to the buoyant uplift of the groundwater for average density rock (S.G. = 2.65).

The importance of water pressure control is shown by Figure 2 whereby Hoek (2) relates the resultant increase in slope angle, due to complete depressurization of a saturated slope to the primary factors which determine the slope angle.

Figure 3 relates mass permeabilities to recommended water pressure control methods (Figure 4), in order of increasing cost. The information is condensed from Brown (3) and Skempton (4).

Figure 3 shows that vacuum wellpointing is a viable method for draining lower permeability soils, (fine silts). Vacuum wellpointing (5) is a proven system for de-watering saturated soils. It employs the principle whereby a column of water, exhausting to a vacuum, is able to be raised 10.3 m (MSL) (33.9 ft). This is due to the pressure exerted by the atmosphere on to the surface of the liquid outside of the enclosed area (Figure 5A). Wellpoints (Figure 5B) are smaller diameter wells that are fully isolated from the atmosphere other than at the point where the water enters the system. An air separation chamber continuously removes air from the system with the water being discharged so as not to interfere with the vacuum unit. The maximum depth of lowering a water table with one stage of wellpoints is approximately 5 m (5). This differs from the above (10.3 m) due to the impossibility of producing: (1) a total vacuum, and (2) a frictionless drainage system.

This paper evaluates the practicability of developing a vacuum horizontal drain. The vacuum assisted drain would result in an increased hydraulic gradient due to exit pressures being below atmospheric. Additional benefits would arise from increased rates of drainage, the ability of depressurizing lower permeability materials, and being able to dewater below the level of the drains. Vacuum drainage below a failure surface would change the direction of the seepage forces so that they would act approximately perpendicular to the failure plane and as a consequence, increase stability. Three case histories are represented which depict the use of vacuum horizontal drainage.

CASE HISTORIES

The following are three brief case histories where vacuum drainage has been employed for slope depressurization.

CASE 1 TABLE TUNNEL - BRITISH COLUMBIA RAILWAY

This is the most recent application of vacuum drainage and consequently the most refined. This case history will emphasize the technique of application whereas the remaining two will concentrate on the effects of vacuum drainage.

The project was located in north eastern British Columbia, 125 km north of Prince George (Figure 6). During the excavation of the west portal of the table tunnel, (Figure 7), a small slide in lacustrine silt occurred above the 1195 m elevation. The slide was confined to an area within the excavation boundaries and remedial works involved regrading to remove slide debris. Upon further excavation to subgrade elevation, the high in-situ moisture conditions, (15 to 20 percent), plus overall slope dimensions (150 m long by 50 m high) raised questions concerning the long term, overall slope stability. It was decided that a consultant would be selected to analyze the overall stability of the slope. While this study was being undertaken, a horizontal drainage program would be undertaken to provide immediate relief to high groundwater pressures near the face of the cut slope. Water pressure levels ranged from 1.6 m to about 7.0 m below ground surface. Steps were taken to drill vertical test holes and to install piezometer and inclinometer casings. Immediately following this program, a horizontal drill was mobilized, complete with all vacuum dewatering equipment necessary to depressurize the slope. Drilling was carried out between March 31 and April 8, 1982 and vacuum applied until May 13. Four drill pads were located along the base of the slope with each hole approximately 30 m deep. A summary of the vacuum flow rates is shown in Table 1.

Summarized Subsurface Conditions

The cut slope was generally formed in a silt to clayey-silt stratum containing lamina to thin layers of fine sand to silty-fine sand. Surficial fill deposits of variable thickness were generally overlying the silt. Underlying the compact to very dense/stiff to hard silt, at a somewhat steeper slope angle than the cut slope, a stratum of very dense sandy till was encountered. Both the silt and the till contained cobbles and boulders.

Water level pressures in the silt and till were variable; some artesian pressures were noted in the till at the toe of the slope. Pressure levels in the silt appeared more depressed in the eastern (Sec. B) and downslope western piezometers. (Sec. A)

Technique of Application

A schematic representation of the "vacuum system" is shown by Figure 8. The horizontal drain consists of a perforated section plus a grouted section which is to serve as a seal. The length of the grouted section is critical since it must "seal" any fissures that may penetrate to surface and consequently reduce the effectiveness of the vacuum. The vacuum pump, Figure 9, was capable of exhausting 12 l/sec of air and producing 0.7 m (Hg) of vacuum at mean sea level. A header system was employed whereby the four pads were connected to the vacuum pump through valves enabling a vacuum to be applied individually or in combination with each other. The exhausted water flow could be measured since flow to the pump exited through a port. The vacuum was monitored through the reading of a gauge attached to the unit. The pump was a diesel-driven, self-contained unit capable of exhausting a constant volume.

Borings were advanced using the rotary technique with clear water as circulation medium. Mill tooth tricone rock bits were employed to cut the formation, in grades of medium-soft, medium and medium hard, 9.8 cm to 11.4 cm in diameter. Rate of advancement varied widely with consistency of the geologic section. The 3.2 cm I.D. Schedule 80 PVC was installed using the patented "Aardvark Method" of drain installation, Figure 10. Briefly described, this method involves the use of adequate I.D. rods through which the plastic screen and packer system is installed. The bit is then dropped and the rods are withdrawn leaving the plastic pipe and packer in place. Following the retraction of the drill rods, the packer-seal was installed.

The seal of the annulus between the boring wall and the drain pipe is considered of utmost importance. For this reason, a special packer system was designed to (1) allow installation using the Aardvark Method and (2) provide an effective basis for vacuum application. The packer system was constructed of mandrell-laid rubber, slid over and banded to 3.2 cm PVC, Figure 11.

The inflation tubing had a check valve at the outer end of the drain pipe, between the pipe and the nitrogen tank so the packer could be inflated, and the nitrogen bottle disconnected leaving the packer inflated to 172 KPA. Grout consisting of Type II cement was then pumped into the boring via a tremie pipe. The inflation tube and check valve was of such length that it could be tucked into the vacuum manifold, and re-inflated should deflation occur.

Conclusions

Due to the discontinuous nature of the groundwater system, there was no appreciable drop in the piezometric pore water pressure. The drains probably intersected sand seams and provided some depressurization locally, however, these seams were isolated from the piezometers. The drains are all producing water presently, under gravity drainage, and therefore are not impaired as a consequence of vacuum experiments. The opinion at B.C. Rail is that the drains are being effective in providing a drawdown of the phreatic line. During vacuum initiation the rate of drawdown was probably accelerated as demonstrated by the increased flow rates.

CASE II GRANITE LAKE PIT, GIBRALTAR MINES

The Gibraltar mine is located in the south-central portion of the province of British Columbia, approximately 370 km north east of Vancouver, Canada, Figure 6. The most important geological features with respect to the groundwater problems at Gibraltar are fracture systems, joint sets, shear zones and gouge filled faults. The first three provide the open spaces necessary for the storage and movement of groundwater whereas the gouge-filled faults usually act as damming structures. In general the rock at Gibraltar is a highly fractured and jointed, altered quartz diorite. The main ore minerals are chalcopyrite and molybdenite.

Location and Program Layout

The vacuum was installed on the east wall of the Granite Lake pit of Gibraltar Mines Ltd. and tested for a period of two weeks under field conditions during May 1981. The purpose of the test was to determine:

(1) Whether a vacuum can be developed in a fractured medium

(2) What effect vacuum has on the groundwater regime.

The area selected had an average hydraulic conductivity of 10^{-7} m/sec in a moderately fractured rock slope, Figure 12. The study area was limited to a 13.7 m high bench. Three 15.2 m - 7.6 mm diameter vertical holes spaced approximately 15.2 m apart were drilled for the purpose of installation of standpipe piezometers. Three horizontal drains approximately twenty-one metres long were inclined at +2 percent and drilled from the bench below to be located midway between the previously drilled piezometers. A seal 6.1 m long was employed which was determined to be the extent of the highly fractured zone. This was required in order to ensure a vacuum tight system. The vacuum pump and seal were similar to that described previously (case study I). The piezometers were subsequently monitored in order to determine the effect that the horizontal drains would have on the water regime. The second stage required the system to be placed under vacuum with simultaneous monitoring being conducted. Further details may be found in a paper by Brawner and Pakalnis (6).

Results

The three drainholes were initially dry and subsequently the area was recharged to the level indicated by Figure 13 whereupon vacuum test I was conducted. Figure 13 shows the elevation of the three drains relative to the piezometric level. Upon the initiation of vacuum the following was observed:

- 380 mm (mercury) of vacuum recorded on the central and southern drain respectively

- flow increased from 0 l/s to 0.2 l/s which was constant for fifteen minutes whereafter no flow occurred

- the central and southern piezometric levels dropped by 0.15 m.

It must be noted that the water table was below the level of the drains. However, the vacuum resulted in the lowering of the phreatic surface.

The area was further recharged until the water levels were as indicated by Figure 13. The observations were as follows:

- a vacuum of 178 mm, 165 mm, 381 mm was achieved on the south, centre and north drain respectively

- upon the initiation of vacuum, flow increased from 0 l/s to 0.3 l/s, which diminished to a no-flow condition thirty-two minutes later

- the above resulted in an immediate drop in the water table. The centre and south piezometric levels were lowered by 1 m.

CASE III: MODEL TESTING

A laboratory model was constructed (Figures 14 and 15), having dimensions 120 cm x 215 cm x 45 cm high. Horizontal drains, 50 cm in length, were installed which were composed of a 15 cm perforated segment of 0.6 cm ID PVC pipe. The model was filled with 30 mesh sand which had a permeability of 10^{-2} cm/sec. A 1:1 slope was constructed with a recharge and discharge pond 45 and 15 cm in height respectively. Piezometers (0.6 cm ID) were installed in order to monitor the effect of drainage on the water table

Results of Model Testing

A comparative study between vacuum assist and gravity drainage was conducted by monitoring the drawdown rates within the individual piezometers. The model was recharged to a base situation and subsequently analyzed under the respective drainage conditions. Relative drawdown is shown by Figure 16.

PRACTICAL IMPLICATIONS

The prototype that was developed is applicable to a pit depressurization system since it is a self-contained unit that has been technically proven by use in the wellpoint industry throughout the past thirty years. The system has application in the following areas:

- Horizontal drains are effective in rocks having a mass permeability greater than 10^{-6} cm/s. Vacuum assist drainage is able to draw materials of lower permeabilities as indicated by wellpointing (10^{-8} cm/s). (Figure 3)

- The system tends to accelerate the rate of drawdown which is an important factor in stabilizing active slides. The reason being that with increased deformation due to slide movement, the shear strength is reduced. Consequently vacuum assist drainage results in less strength loss.

- Vacuum assist drainage enables an individual drain to exhibit a wider zone of influence. This is partly due to an increased permeability; the capilliary effects within the joints are partially negated by the increased hydraulic gradient, (Figure 17). In addition, the water table can be drawn a further 5 m below the level of the drain. Consequently fewer drains have to be installed in order to produce benefits similar to those of gravity drains.

- Vacuum assist drainage may be employed in sub-surface galleries in order to assist drainage in the stabilizing of major slides.

- Vacuum drainage re-directs seepage forces so that they act perpendicular to the failure surface. This is shown by Figures 18A and 18B whereby the drain is below the failure plane. The consequence is that the normal load acting on the failure plane is increased and, as a result, the shear strength of the material is increased.

- Vacuum assist drainage may be employed in depressurization of waste embankments, tailings dams, soil landslides and underground backfill.

SUMMARY AND CONCLUSION

It is evident from field and laboratory observations that:

1. A vacuum can be created in a fractured media. 381 mm (mercury) of vacuum was created at Gibraltar Mines Ltd.

2. Vacuum assist drainage increased the rate of drawdown. Laboratory investigations suggest a two-fold rate of increase.

3. Vacuum drainage results in immediate drawdown. Laboratory investigations indicate that the greatest drawdown occurs in the first minutes of operation.

4. Vacuum drainage enables a drawdown below the level of the horizontal drains. This was shown by field investigations whereby vacuum assisted horizontal drains, located 2.3 m above the water table, had the subsequent effect of lowering the phreatic surface by 15 cm.

5. A direct relationship existed between the amount of vacuum and the subsequent drawdown.

It was concluded from the study that vacuum horizontal drains are a viable technical addition that can efficiently augment gravity systems.

REFERENCES

1. BRAWNER, C. O. The Influence and Control of Groundwater in Open Pit Mining. Fifth Canadian Symposium on Rock Mechanics, University of Toronto, 1968.

2. HOEK, E. AND BRAY, J. W. Rock Slope Engineering. Revised Second Edition, Institution of Mining and Metallurgy, London, 1977.

3. BROWN, A. The Influence and Control of Groundwater in Large Slopes. Third International Symposium in Open Pit Mining, Vancouver, B.C., 1981.

4. SKEMPTON, A. AND GLOSSOP, R. Particle Size in Silts and Sands. Journal of the Institution of Civil Engineers, Volume 25, London, 1945.

5. TERZAGHI, K. AND PECK, R. Soil Mechanics in Engineering Practice. Wiley, New York, 1948.

6. BRAWNER, C. O. AND PAKALNIS, R. Vacuum Drainage to Stabilize Rock Slopes on Mining Projects. First International Mine Water Congress, Budapest, Hungary, April 1982.

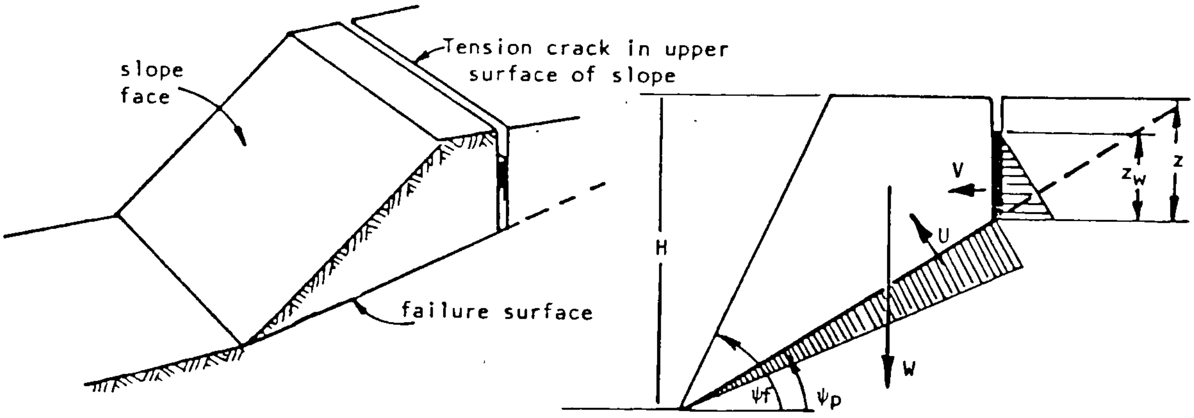
Acknowledgements:

The authors wish to thank Doug Carabetta, Geotechnical Engineer, British Columbia Railway for the information he provided for the first case history for this paper.

TABLE 1 - HORIZONTAL DRILL HOLE SUMMARY

DRAIN NO.	LENGTH (m)	SEAL (m)	GRAVITY FLOW	VACUUM	COMMENT
1-1	30.5	9.1	DRY	TRICKLE	)
1-2	30.5	9.1	.2 l/min	SLIGHTLY >0.2 l/min.	) COMBINED) VACUUM OF) 63.5 cm
1-3	7.6	1.5	TRICKLE	-	)
1-4	22.9	7.6	-	-	NO VACUUM
2-1	12.2	6.1	-	-	12.7 cm Hg
3-1	19.8	4.6	1.5 l/min	2.4 l/min	15.0 cm Hg
3-2	33.5	9.1	0.5 l/min	-	NO VACUUM
3-3	15.2	3.0	TRICKLE	_	NO VACUUM
3-4	32	7.6	0.61 l/min	9.0 l/min	18 cm Hg.
4-1	33.5	9.1	TRICKLE	Not Record.	N.R.
4-2	45.7	6.1	.7 l/min	N.R.	N.R.
4-3	41.1	4.6	N.R.	N.R.	N.R.
4-4	38.1	7.6	N.R.	N.R.	N.R.

HOLES DRILLED +3%



$$F = \frac{CA + (W \cos\psi p - U - V \sin\psi p) \tan\phi}{W \sin\psi p + V \cos\psi p}$$

WHEREBY:

C= cohesion on failure plane

W = weight of failure block

ψ p= inclination of failure plane

ϕ = friction angle on failure plane

U = buoyant force of water

V = lateral force exerted by water within tension crack

FIGURE I INFLUENCE OF WATER PRESSURE ON SLOPE STABILITY

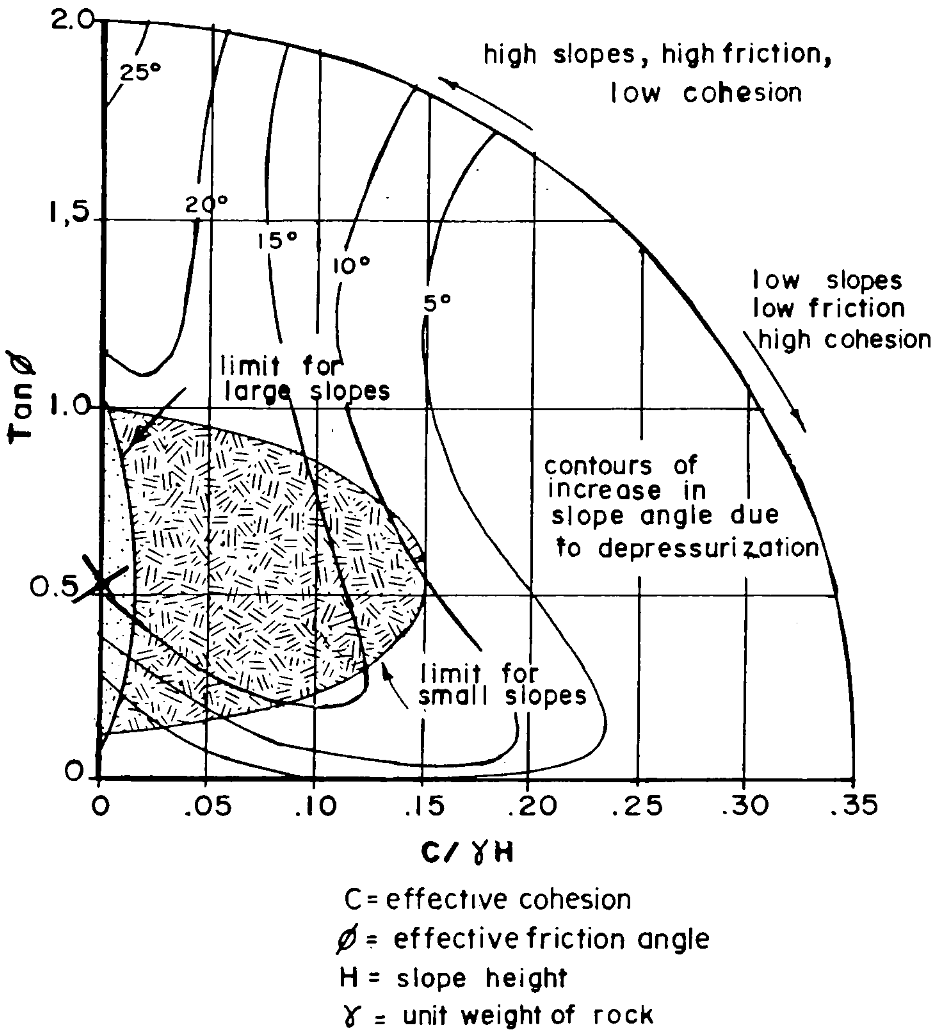


FIGURE 2 RESULTANT INCREASE IN SLOPE ANGLE DUE TO TOTAL DEPRESSURIZATION

(AFTER HOEK) [2]

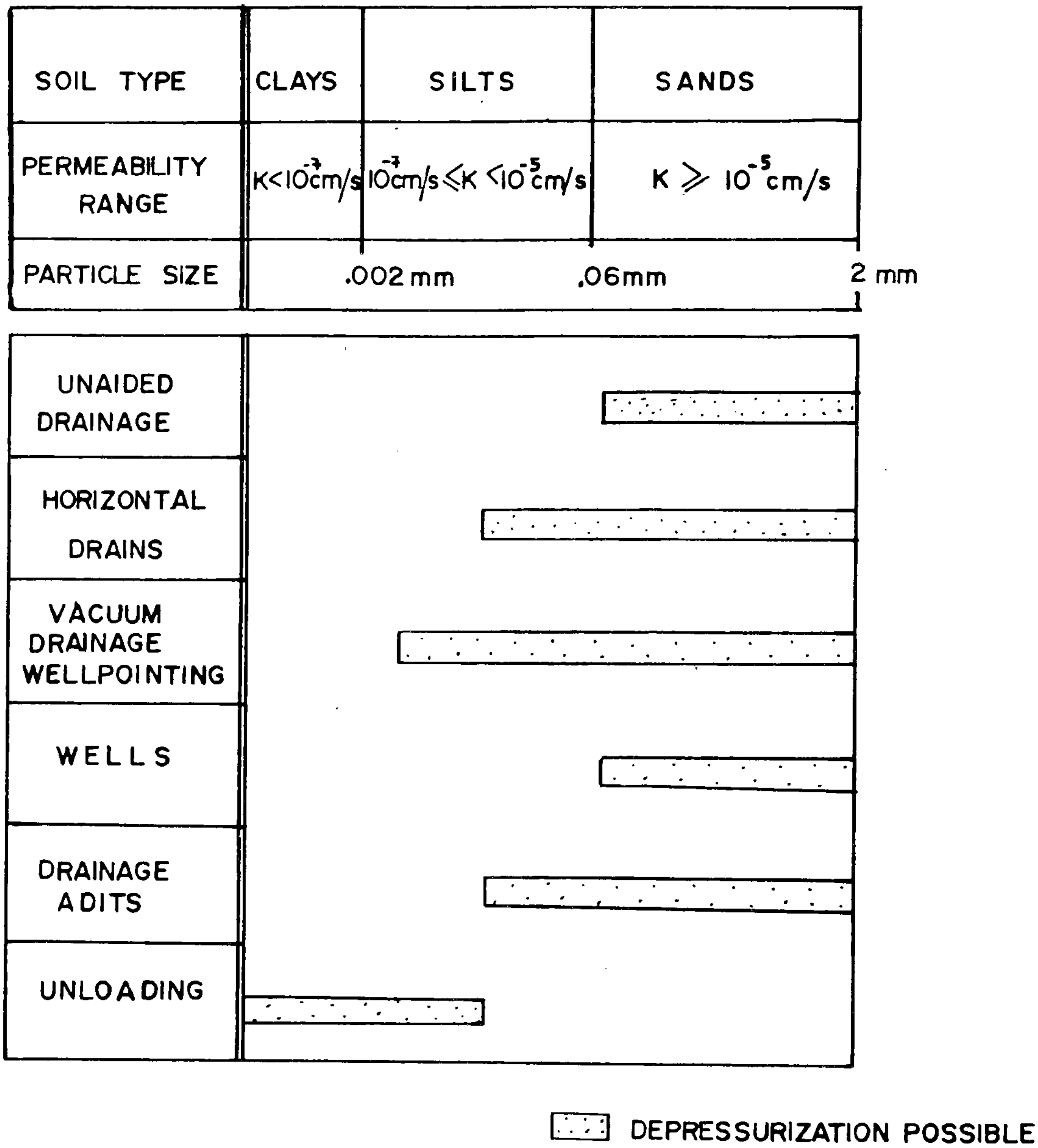


FIGURE 3 RECOMMENDED DEPRESSURIZATION TECHNIQUES FOR VARIOUS MASS PERMEABILITIES (AFTER BROWN [3] & SKEMPTON [4])

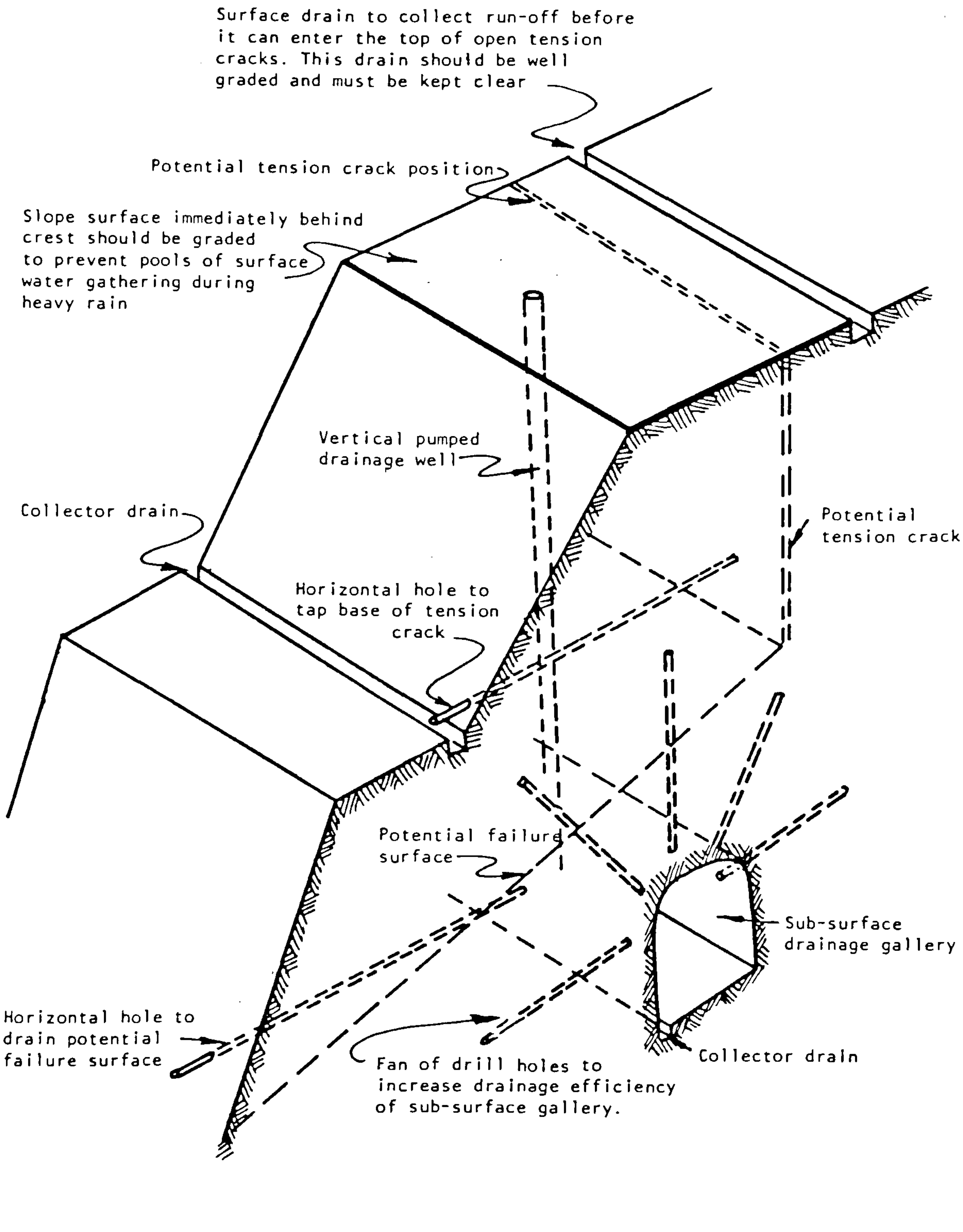
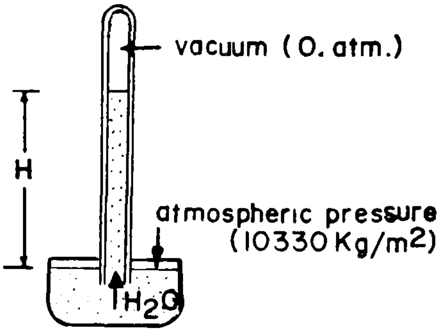


FIGURE 4 SLOPE DRAINAGE SYSTEMS (AFTER HOEK) [2]



WHEREBY:

H = height of water column

$$H = \frac{\text{atmosheric pressure } (10330\,Kg/m^2)}{\text{density of water } (1000\,Kg/m^2)}$$

$$= 10.3\,m \quad \text{(at mean sea level)}$$

FIGURE 5A VACUUM PRINCIPLE

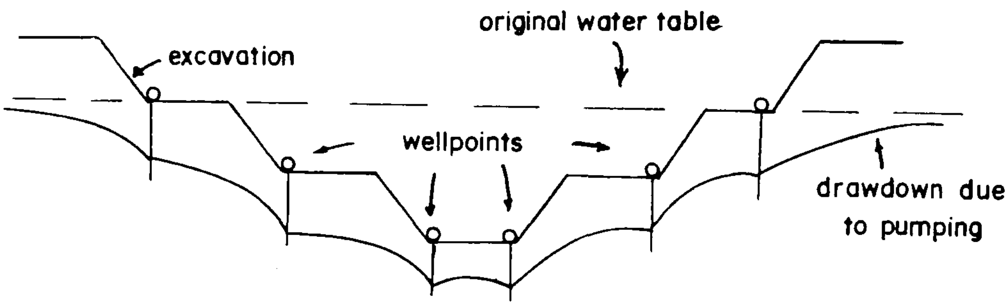


FIGURE 5B VACUUM WELLPOINTING APPLIED ON 5m. LIFTS

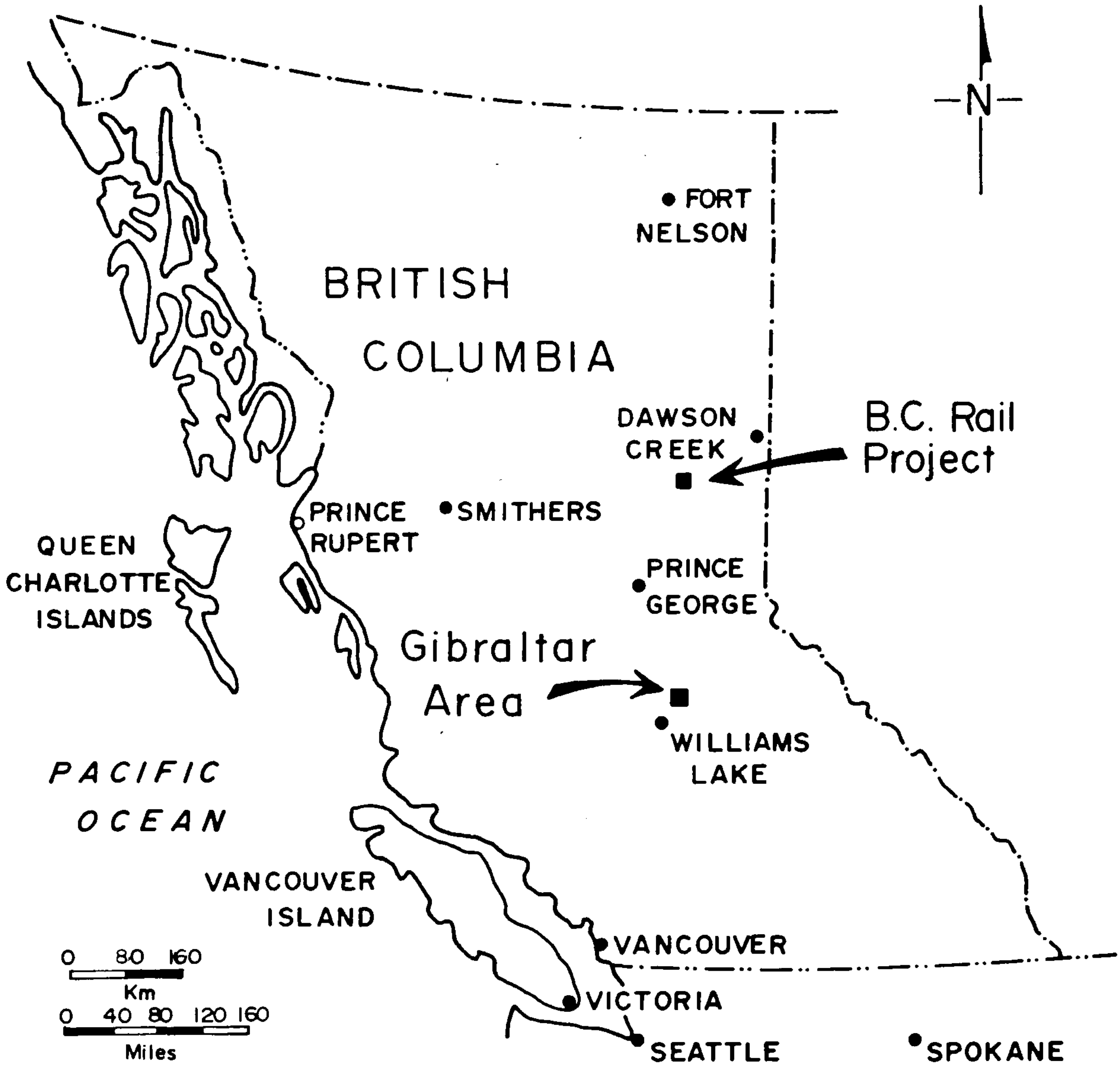


FIGURE 6 LOCATION MAP OF CASE STUDIES

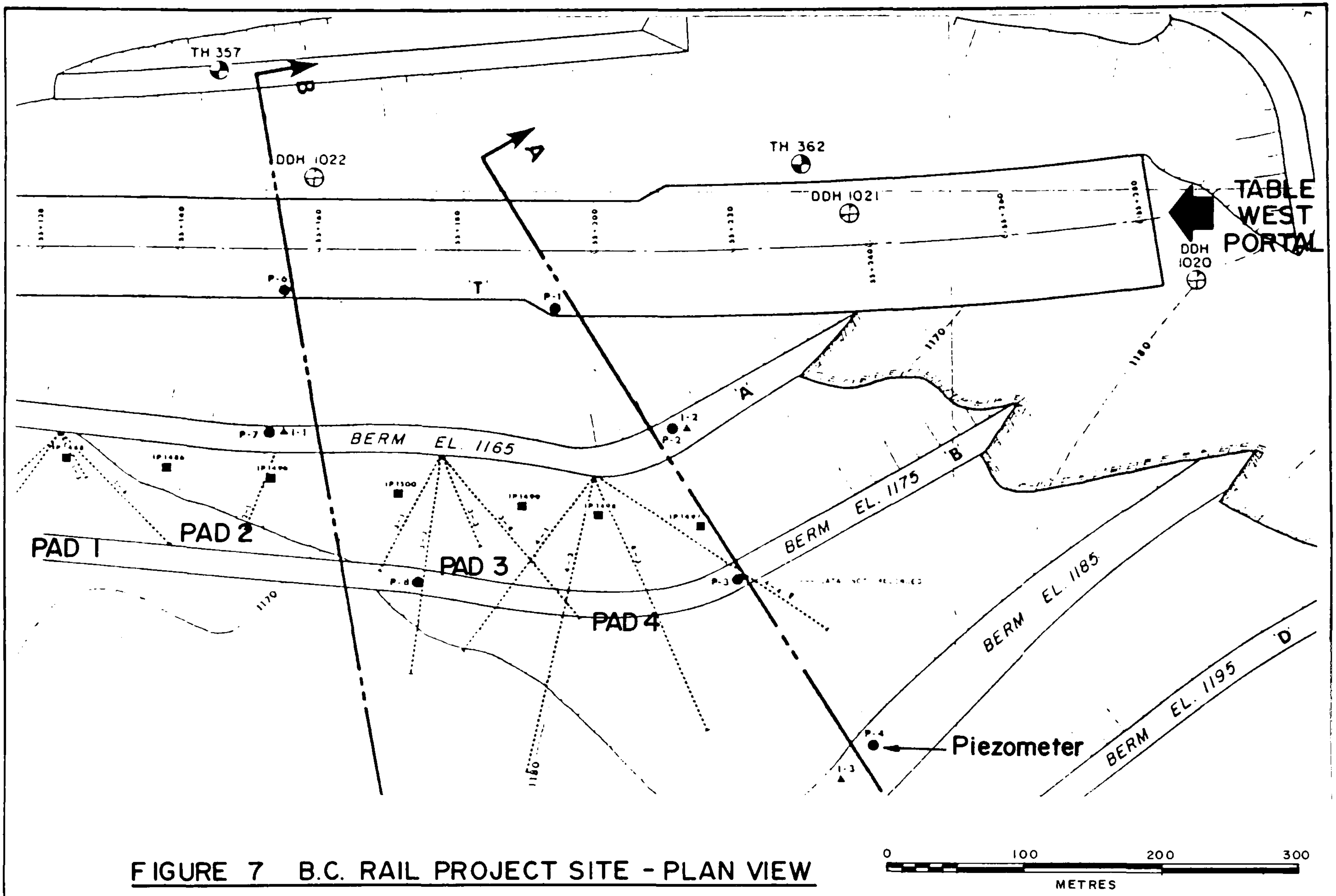


FIGURE 7 B.C. RAIL PROJECT SITE - PLAN VIEW

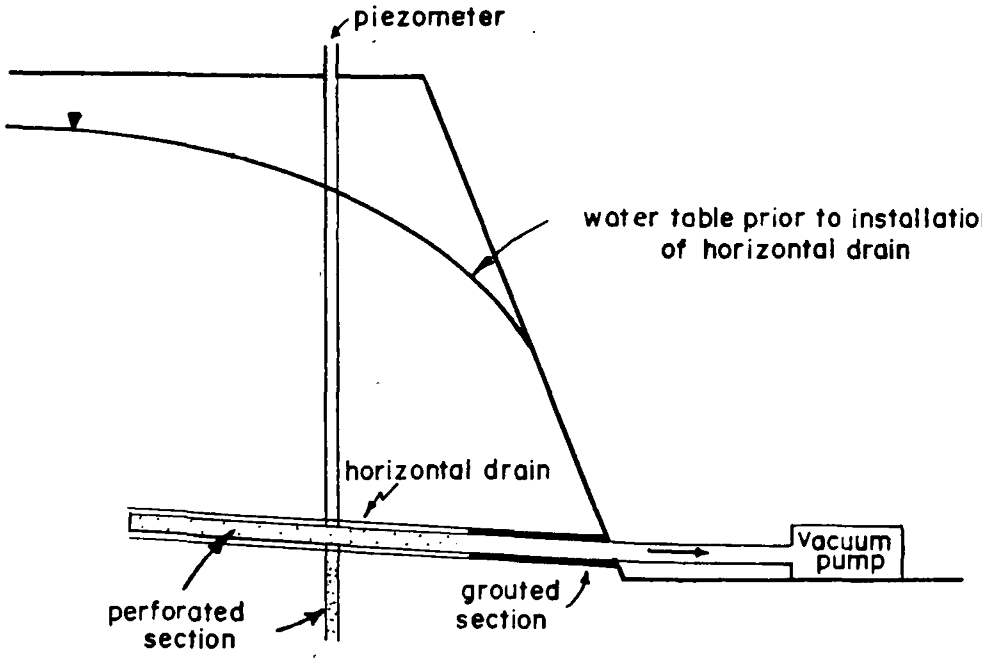


FIGURE 8 SCHEMATIC REPRESENTATION OF VACUUM SYSTEM

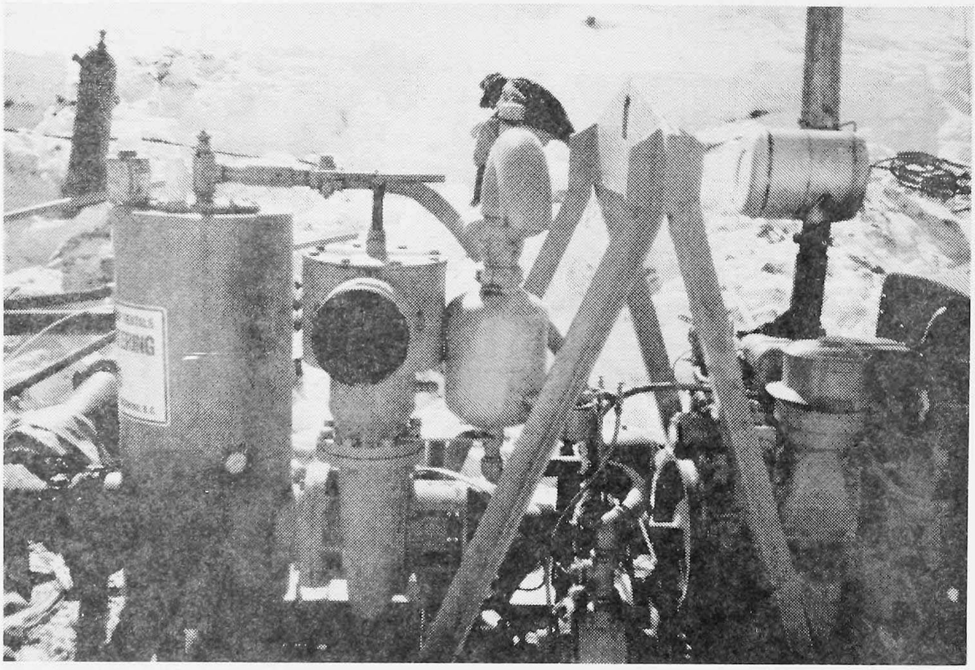


FIGURE 9 VACUUM PUMP DEPICTING - EXIT PORTAL
- VACUUM GAUGE

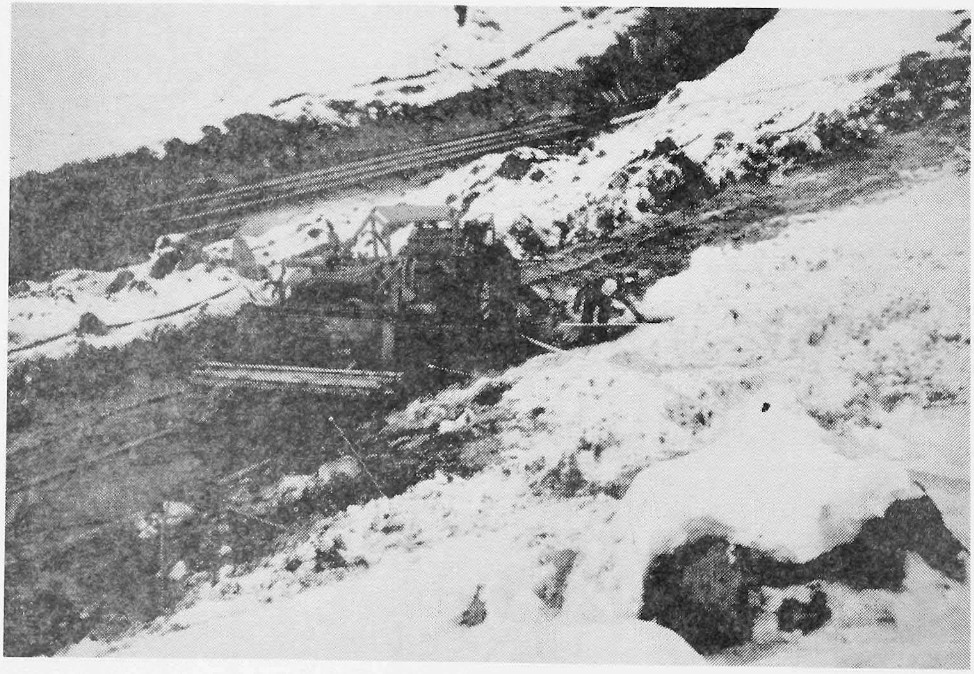


FIGURE 10 INSTALLATION OF VACUUM DRAINS

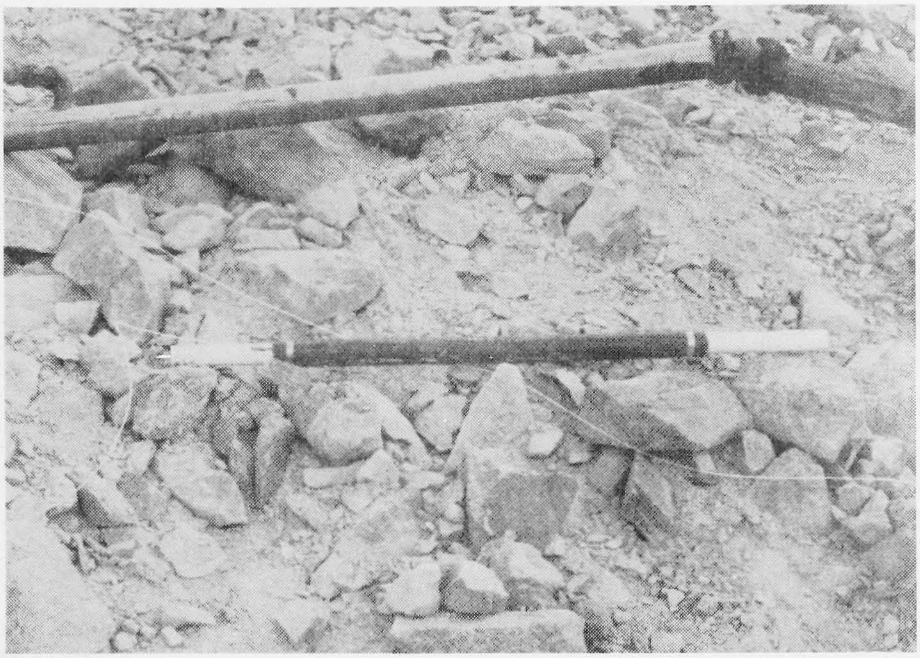


FIGURE 11 INFLATABLE SEAL

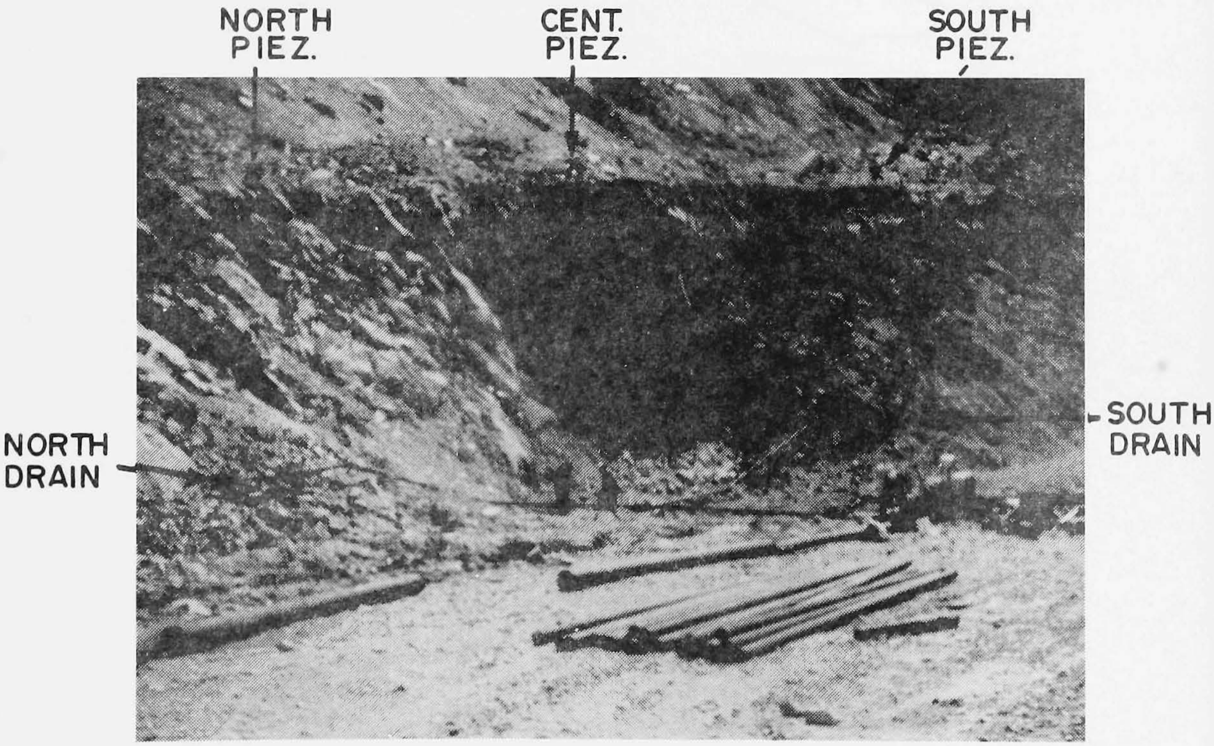


FIGURE 12 GIBRALTAR TEST SITE

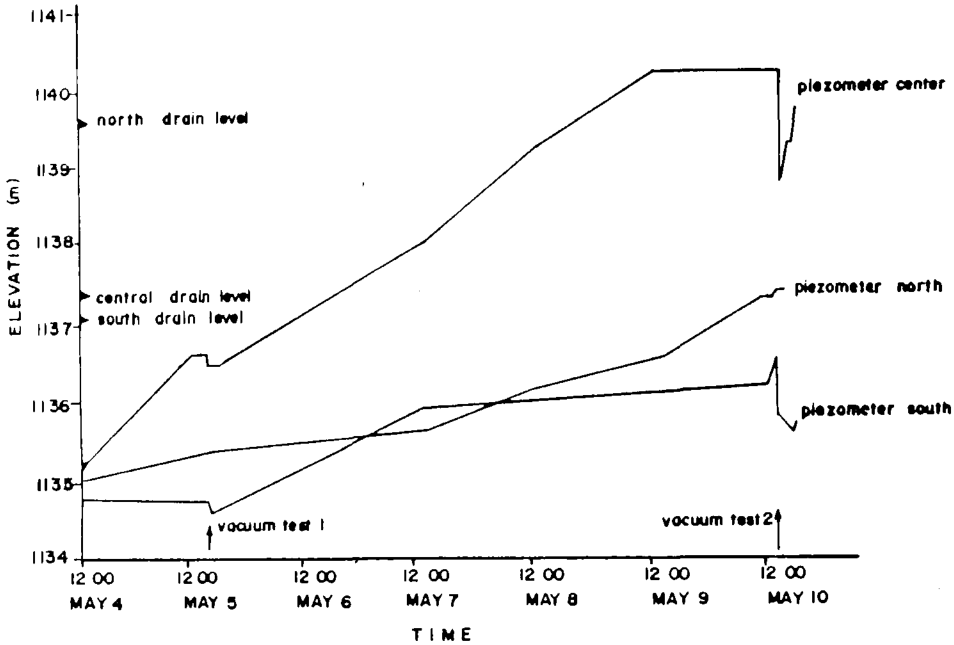


FIGURE 13 WATER LEVEL VERSUS TIME

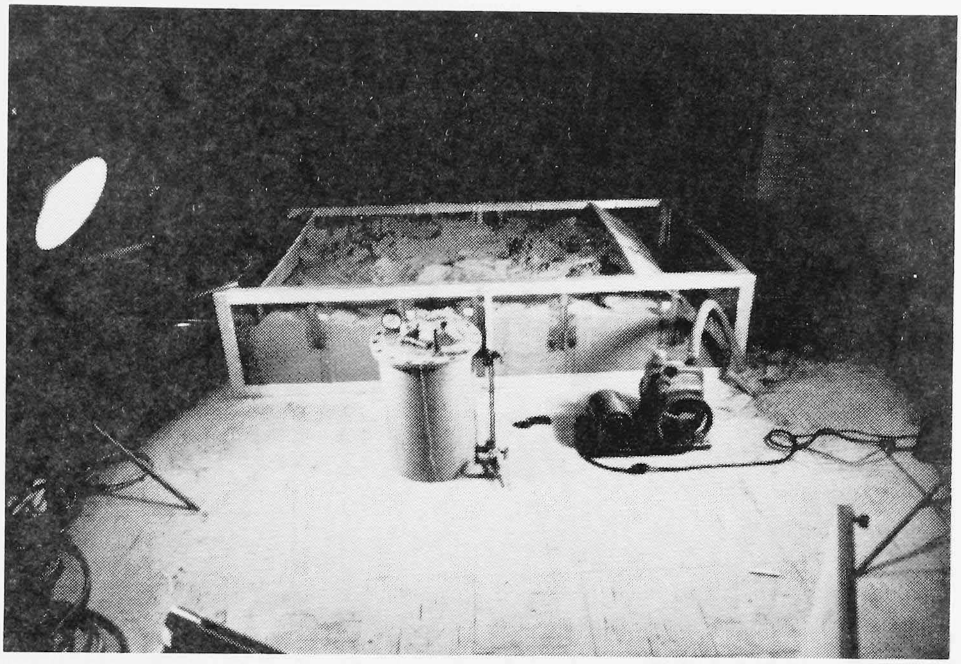


FIGURE 14 HORIZONTAL DRAIN MODEL

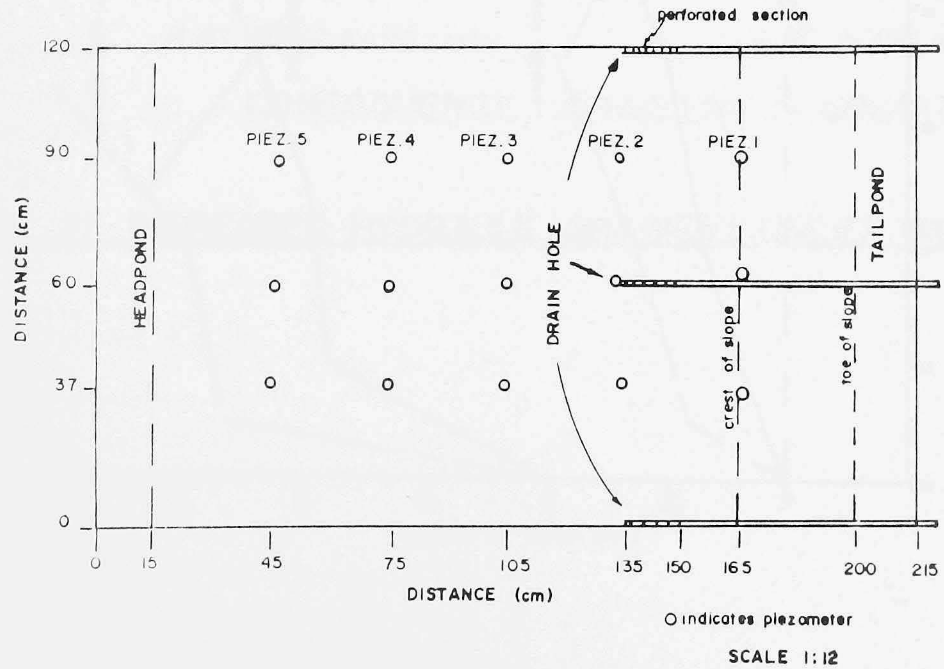


FIGURE 15 PLAN VIEW OF DRAINAGE MODEL

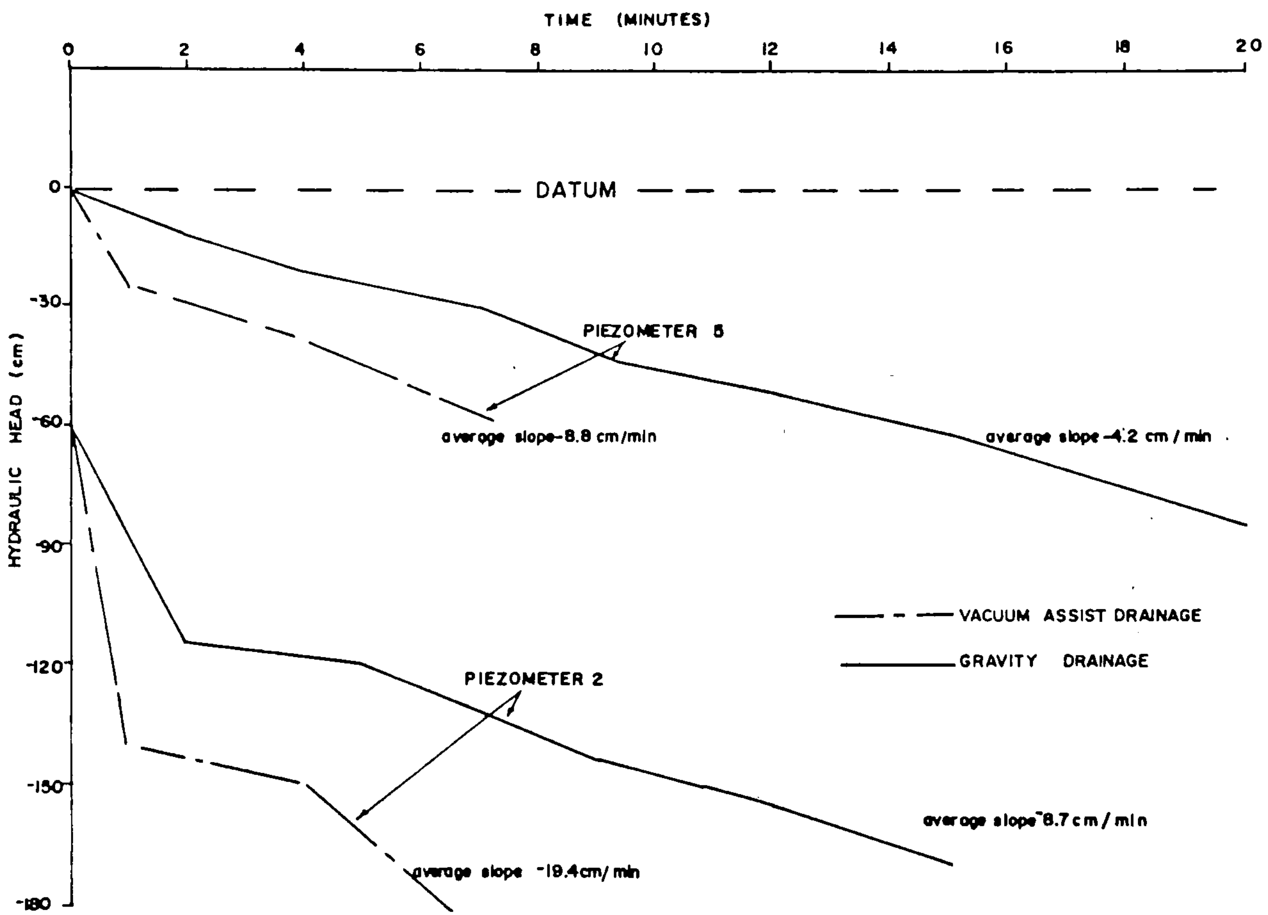
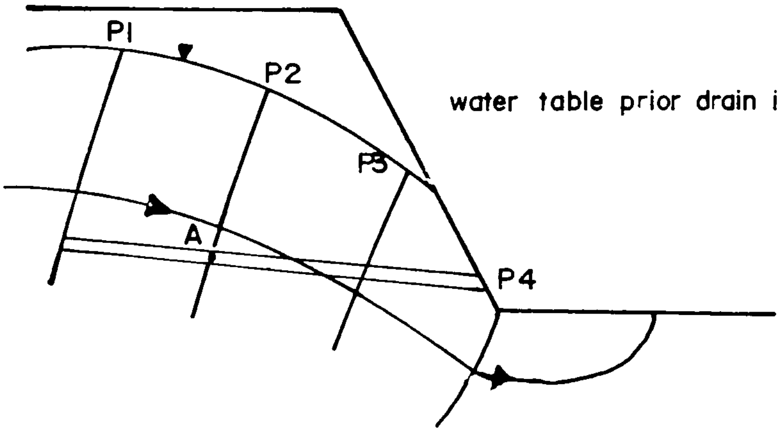


FIGURE 16 PLOT OF DRAWDOWN VS. TIME UNDER: a) gravity drainage
b) vacuum drainage



CASE 1
ASSUME NO VACUUM (P4= 1atm.)
H AT POINT A= P2 - 1atm.

CASE 2
ASSUME VACUUM ASSIST
H AT POINT A= P2 - 0atm.

CONSEQUENCE : GRADIENT IS GREATER:CASE 2

FIGURE 17 INCREASED HYDRAULIC GRADIENT UNDER VACUUM

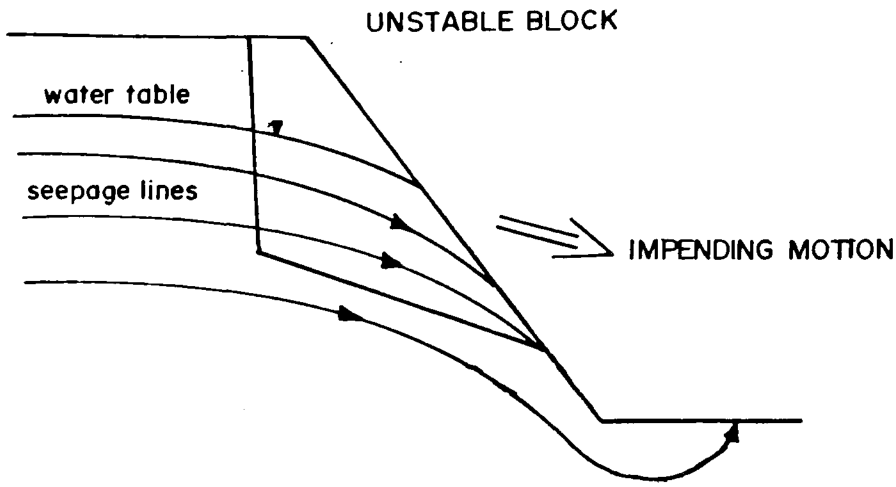


FIGURE 18A SEEPAGE FORCES REDUCING STABILITY

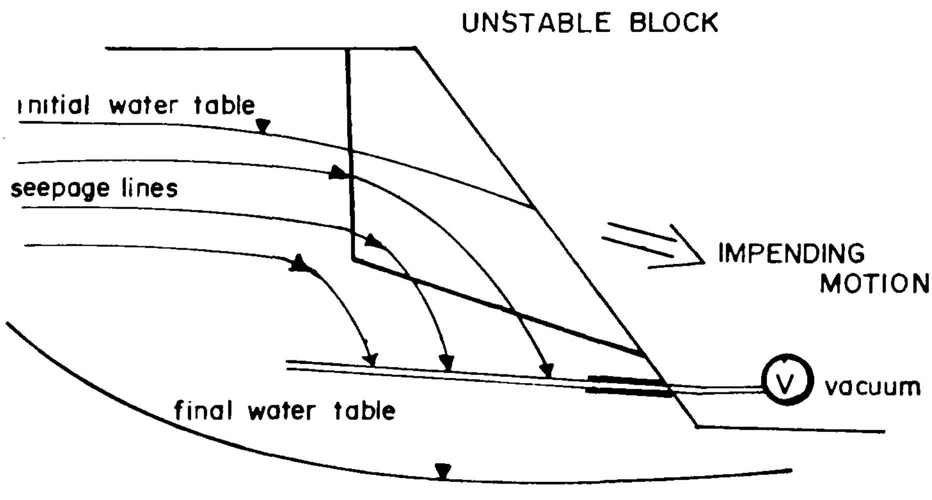


FIGURE 18B SEEPAGE FORCES INCREASING STABILITY

STABILIZATION OF TOPPLING ROCK SLOPE FAILURES

by

Duncan C. Wyllie and David F. Wood

Golder Associates

ABSTRACT

The paper describes the mechanism of toppling failures and alternative stabilization techniques for these failures in relation to rock cuts for highway construction. Toppling failures in rock slopes occur where the slope alignment is approximately parallel to the strike of a regular set of planar geological discontinuities, such as bedding plane fractures or joints, which dip into the slope at a steep angle. This geometry produces a series of thin slabs or blocks, which may topple from the slope onto the highway if the centre of gravity of such a slab falls outside its base. Stabilization of toppling slopes involves changing the geometry of the slabs by such measures as reducing the slope height, flattening the slope face, or bolting slabs together, or by other methods such as supporting the toe, or realigning the slope.

This paper also describes a 130 ft. high toppling rock slope failure which was stabilized by constructing a compacted gravel toe berm. The berm incorporated a gabion wall which formed a ditch to catch rock falls.

INTRODUCTION

The stability of toppling failures can be studied using physical models [1,2,3] and numerical models [4,5], although these analyses can be time consuming and the required facilities may not be readily available. However, Goodman and Bray's limit equilibrium analysis for multi-block toppling failures now allows the analysis of toppling failures and the selection of appropriate stabilization measures to be carried out more readily [6,7]. This paper describes the use of limit equilibrium analysis in examining the stability of toppling slopes, and the design of stabilization measures.

MECHANISM OF TOPPLING FAILURES

Toppling movement can occur in slopes where a regularly spaced set of joints or bedding plane fractures strike parallel, or nearly parallel, to the slope face and dip at a steep angle into the face. This geological structure forms a series of tall, narrow slabs (Figure 1). If the dimensions of a slab are such that its centre of gravity acts outside the base of the slab, then there is a potential for the slab to topple. This geometical criterion is given by the relationship [6]:

$$\text{if} \quad y/\Delta x > \cot\alpha \ \text{then toppling occurs} \qquad (1)$$

where: y = slab height
Δx = slab width
α = dip angle of base of slab

Short slabs at the crest which do not meet the criterion shown in Equation (1) and do not slide on the base, are stable (Figure 1, slabs 5,6,7). A longer slab which topples (slabs 2,3,4) exerts a thrust on the slab immediately below it on the slope. This thrust produces a turning moment on the lower block which increases its tendency to topple creating a "domino effect".

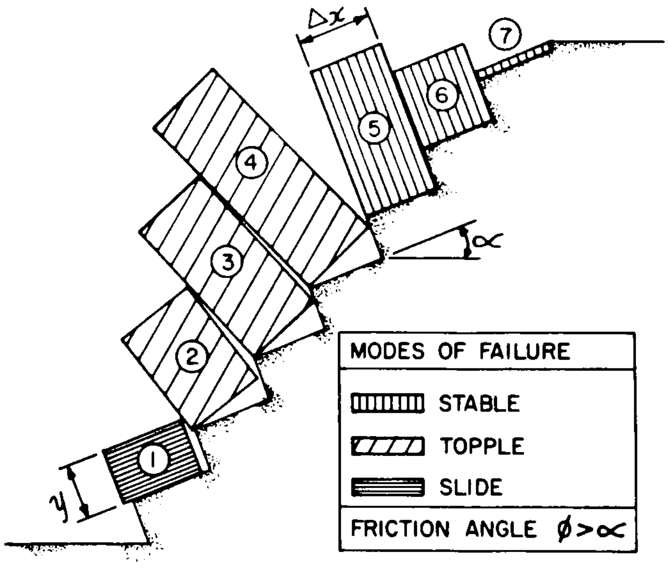


Figure 1 Mechanism of Toppling Slope Failure

However, each slab is constrained from toppling by the force transmitted upwards from the block below it, and from the shear forces acting on its sides. As this motion progresses down the slope, the magnitude of the thrust from the upper slabs may be sufficient, near the toe, to cause short slabs that do not meet the criterion shown in Equation (1) to topple. However, where the moment from the slabs above is such that the ratio of the shear to normal forces on the base of the slab exceeds the friction coefficient of the base, then the slab will slide rather than topple (Figure 1, slab 1). Since the magnitude of the thrust is dependent upon the friction of properties of the surfaces of the slabs, as well as their dimensions, stability is governed both by the geometry of the slope and the rock properties.

The analytical procedure is to define the dimensions of the slope, the geological structure, the rock properties, and the angle of the failure plane. The toppling and sliding characteristics of each slab are then examined in turn starting at the crest to determine the stability characteritistics of the slabs at the toe. This mechanism has been clearly identified in a number of actual failures.

This limit equilibrium analysis can also be used to estimate the factor of safety of the slope in the following manner. Firstly, the friction coefficient of the side and base surfaces of the slab is determined either by laboratory testing or engineering judgement. This is the 'available friction coefficient'. Note that it may be appropriate to use different friction values on the sides and base if, for example, one set of fractures has an infilling with a low friction coefficient. The second step is to carry out limit equilibrium analyses with varying friction coefficients until the condition is found where the toe blocks are just stable. This is the 'required friction coefficient', and the factor of safety is given by the following relationship:

$$\text{Factor of Safety} = \frac{\text{Available Friction Coefficient}}{\text{Required Friction Coefficient}} \qquad (2)$$

STEREOGRAPHIC PROJECTION

A rapid means of examining a slope for potential toppling failure is to use a stereographic projection [6,8]. This can show whether the poles to the planes forming the toppling slabs lie within a certain critical range of combined dip and dip direction where failure is possible.

Figure 2 illustrates a slope cut at an angle of θ °, containing fractures which dip into the slope at an angle of $(90-\alpha)$. Assume that the friction angle of these fractures is θ . In the region of the slope close to the face, there will be a uniaxial stress condition with stress σ_1 , parallel to the slope face, the normal and shear stresses acting on the fractures are:

Normal Stress	$N = \sigma_1 \sin \beta$
Shear Stress	$S = \sigma_1 \cos \beta$
where	$\beta = 90 - (\theta - \alpha)$

For interlayer slip to occur, the shear stress must be greater than the resisting stress given by:

$$\text{Resisting Stress,} \quad R = N \tan \phi$$

i.e., for slip $S > R$

or $\theta \geqslant \phi + \alpha$ (3)

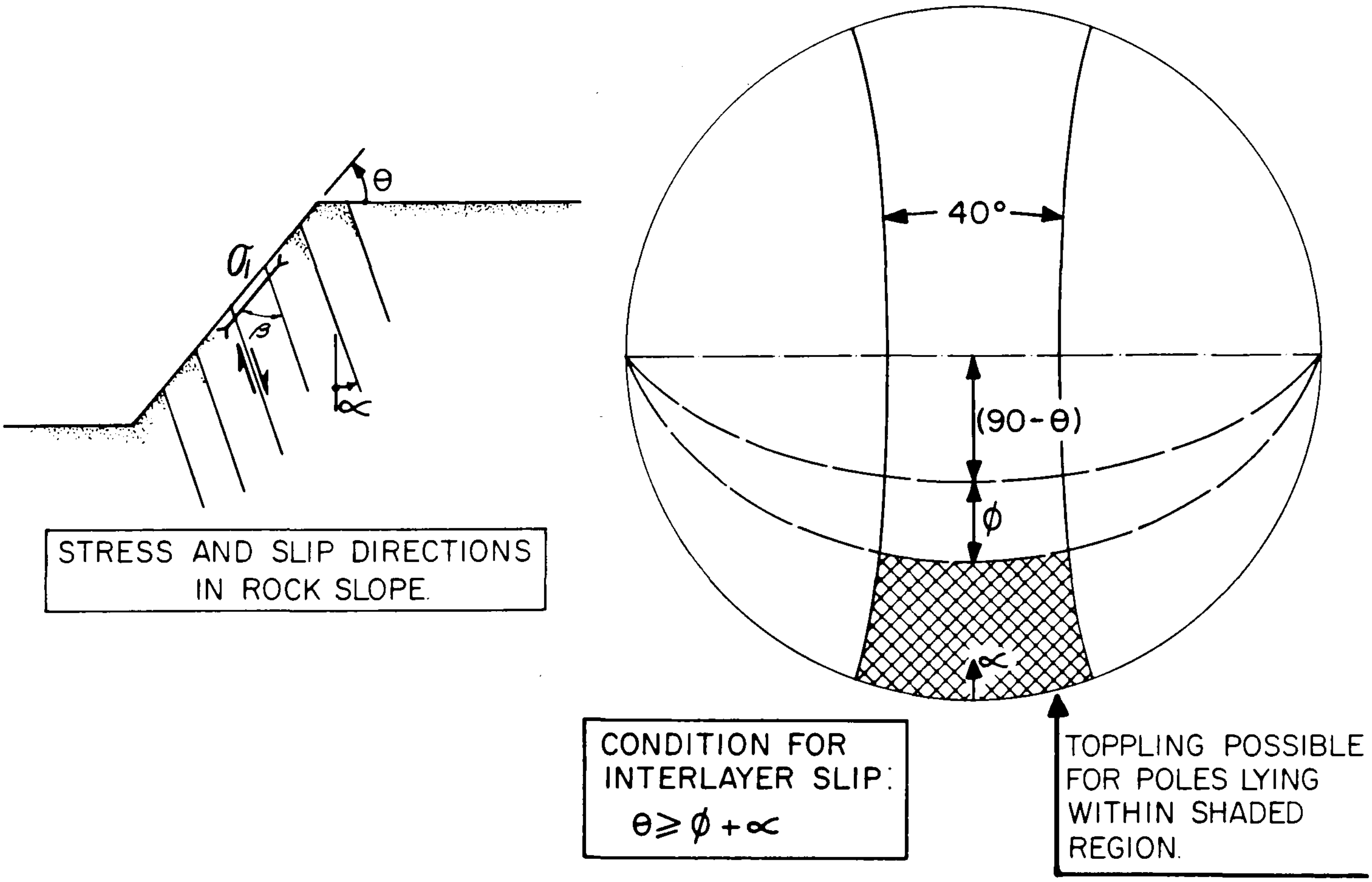


Figure 2 Kinematic Condition For Flexural Slip Which Precedes Toppling

This condition for slip is shown on the stereonet Figure 2 where planes with poles lying within the shaded region, i.e. dips range from $\alpha = 0$ (vertical) to $\alpha = (\theta - \phi)$, have potential for causing toppling. The lateral limits of the shaded zone are governed by the range of strike of the planes in relation to the strike of the cut. Where the strike of these planes differs from the strike of the slope by an amount greater than about 20°, the planes form a series of stepped faces with each slab supporting the one behind it and toppling failure is inhibited. Therefore, the difference between the strike of the fractures causing toppling and the strike of the highway cut will show the approximate lateral extent of the toppling failure (Figure 3). This figure also shows that the smaller the radius of the curve, the shorter the length of the toppling failure.

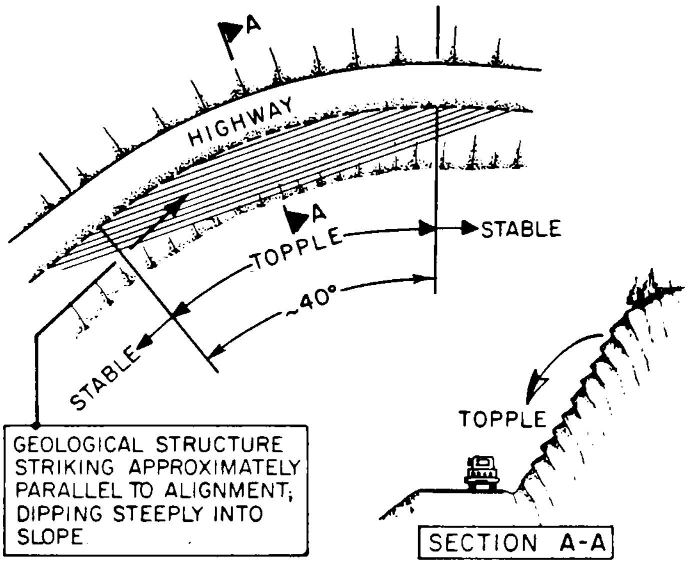


Figure 3 Relationship between highway curvature and toppling failure

STABILIZATION METHODS

The mechanism of toppling failure described in the previous section of this paper can be applied directly to the design of stabilization methods. While the theory may show several methods which are technically feasible, there are usually physical or economic restraints which must be considered in selection of the most feasible methods. For example, in mountainous terrain there is usually little flexibility in alignment, so changing the strike of the slope relative to the geologic structure is rarely possible. Other stabilization methods such as reducing the slope height or angle will depend upon men being able to access the slope and work safely on the face, and upon economic constraints.

The following paragraphs discuss six different stabilization methods for toppling failures, and discuss the relative merits and practicability of each. The techniques are shown diagramatically in Figure 4.

a) Reduce the slope angle to reduce the height (y) of each block. The reduction in slope angle need not reduce the block heights to such an extent that every block does not meet the toppling criterion (Equation 1), but need only be sufficient to reduce the thrust on the lower blocks so that they are stable. The required slope angle to produce a slope with a specified factor of safety can be determined using the relationship given in Equation (2).

   Note that Figure 4(a) shows that the excavation of the face does not extend to the toe of the slope. The small wedge of rock at the base of the slope can be difficult to remove because the width of the working bench becomes limited in this area; the operating width of heavy machinery is about 20 ft. Furthermore, the blocks at the toe of the slope are not sufficiently tall to topple, so there is no need to reduce their height.

b) Unload the slope crest to reduce the height of the upper blocks. In some circumstances, it may be found that a modest reduction in slope height will produce a significant reduction in the thrust produced by the toppling blocks. This resultant thrust at the toe of the slope needs to be reduced by an amount sufficient to prevent the toe blocks from sliding.

   Unloading the crest of the slope can be a somewhat easier operation than reducing the slope angle. This is because in unloading the crest, the excavation can be carried out on one, or possibly two levels, each of which may have an ample operating width. However, in reducing the slope angle, access must be provided to a series of progressively narrower benches. Another factor to consider in both cases is that it may be dangerous to work on the slope if it is very close to collapse.

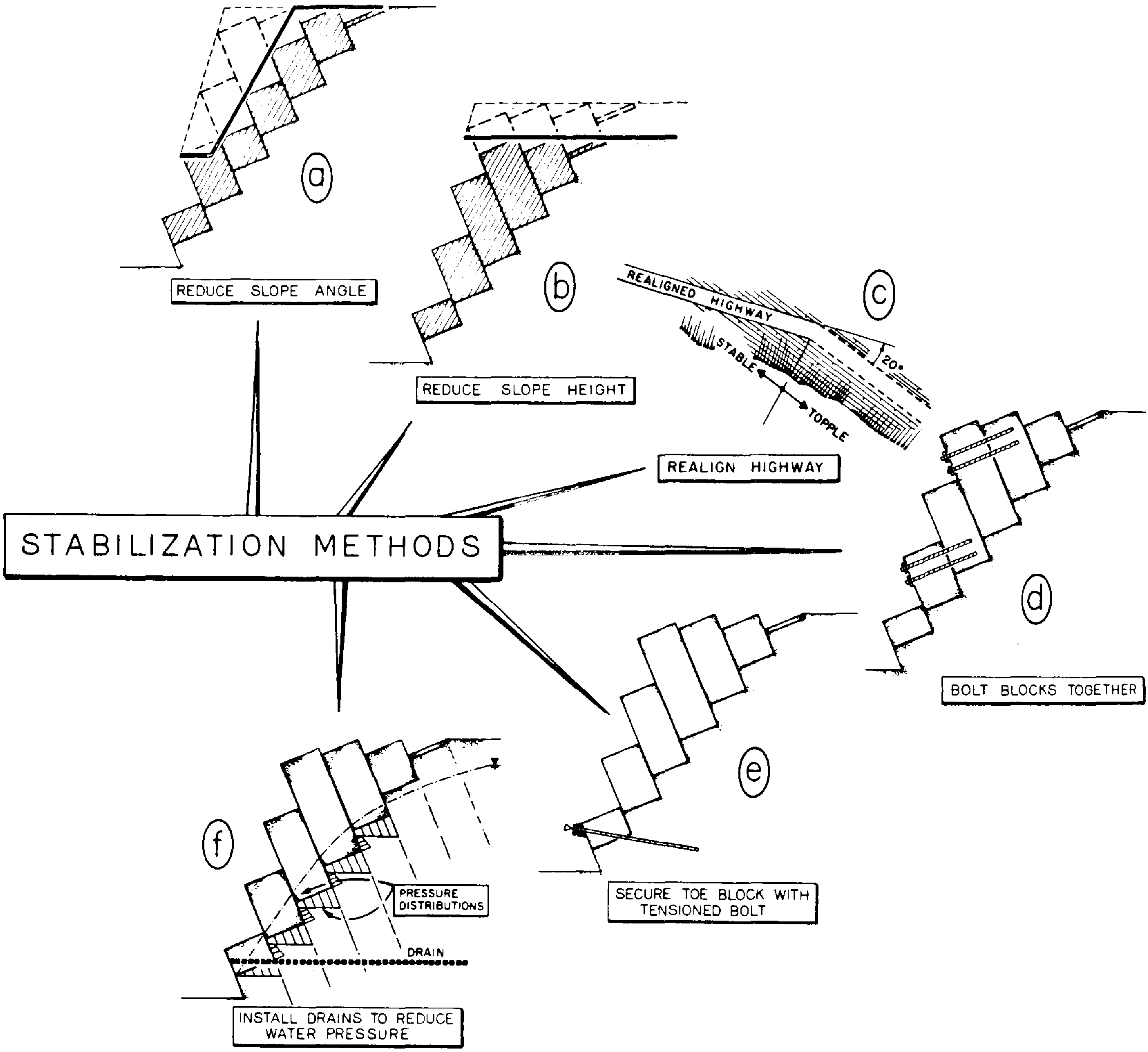


Figure 4 Stabilization Methods for Toppling Failures

c) Realign the highway so that the strike of the cut slope is not parallel to the strike of the toppling fractures. The difference in angle should be about 20° to ensure that continuous faces of toppling slabs are not exposed. Figure 4(c) shows realignment of a tangent, while Figure 3 shows how the radius of curvature influences the length of a toppling slope.

The applicability of this stabilization option will depend upon such factors as right-of-way limits and overall highway alignment restrictions.

d) Increase thickness of slabs ($\triangle x$) by bolting pairs of them together. Apart from changing the geometry of the slabs, the bolts will also improve stability by increasing the resisting force acting between slabs, and prevent interlayer slip (refer to Figure 2). For maximum effectiveness, the bolts should be tensioned so that a normal force between the slabs in increased.

The practicability of this technique depends upon having access to the slope and being able to drill holes and anchor the bolts in rock that has been fractured and loosened by slope movement.

e) Support the toe slabs, whether they are toppling or sliding, by installing tensioned rock anchors through the toe anchored in stable rock. The forces produced by these bolts can be incorporated into the limit equilibrium analysis to give a required friction coefficient for the slope. Thus, a bolting force can be designed to produce a required factor of safety.

Installation of bolts in the toe of a slope is a relatively easy operation because access to the toe is often easy, the bolts need not be particularly long and the required tension is usually modest.

f) Drainage to reduce water pressures is one of the most common means of stabilizing slopes. The pressures acting on the base and sides of blocks reduce the normal force and increase the thrusts respectively acting on the blocks. Thus, water pressure can significantly increase the tendency of a slope to undergo a toppling failure.

Drainage can often be achieved by drilling horizontal drains from the toe of the slope as illustrated in Figure 4(e). If there is little hydraulic connection between the steep fractures forming the toppling slabs, then the drain must be long enough to intersect those fractures which outcrop near the slope crest to ensure good drainage. Piezometers should also be installed to monitor water pressures.

Example of Toppling Failure Stabilization

The concluding part of this paper describes the measures that were taken to stabilize a 130 ft. high toppling failure above a major railroad. The only rock excavation that had taken place during construction of the railroad was near the toe. In this area, the slope angle was about 70° compared to the overall angle of 55°. Also, a 20 ft. wide ditch had formed an overhanging face about 16 ft. high.

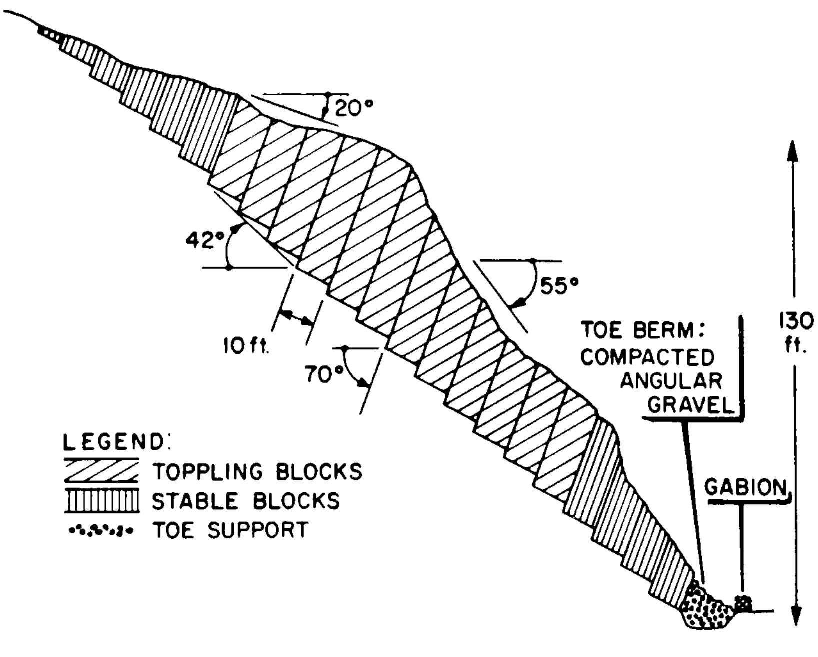


Figure 5 Toe Berm to Stabilize Toppling Slope Failure

The rock type was a moderately strong argillite containing three orthogonal sets of joints. One set, consisting of joints dipping at approximately 70° and spaced about 10 ft. apart, formed a series of slabs dipping into the slope. The other two sets of joints were not as continuous as the first set, but they were oriented such that they often formed the base and sides of the slabs. In general, the geological structure did not form the distinct toppling structure shown in Figure 1, but the slabs were sufficiently well defined to make toppling the most likely failure mechanism.

The hazard to traffic that developed at this site consisted of frequent minor rock falls that were often caught in the ditch, but sometimes reached the track. The cause of these rock falls was considered to be the fracture of rock due to interlayer slip as the first stage of a toppling failure. The rock on the face and the one exposed side of the slope was very loose and broken, indicating some movement was taking place, although no tension cracks, indicating the onset of a major problem, could be found.

A limit equilibrium analysis of the slope was carried out which showed that the required friction angle for a factor of safety of 1.0 was in the range of 25° to 30°. These values were consistent with what would be expected for a fine grained argillite. The analysis was then re-run with these friction angles to determine the support force required at the toe to increase the factor of safety to 1.5. The force was found to be about 3000 lb/lineal foot of slope, which was used in the design of stabilization measures.

The stabilization measures selected had two objectives:

- to provide support at the toe of the slope to limit further toppling motion;
- to provide a catchment area for rock falls.

Support could have been provided by rock bolts, but the broken nature of the rock would have required that a concrete beam be poured along the toe to distribute the bolting force into the rock. This would have been expensive because the site was isolated, and access was difficult. Therefore, it was decided to place a compacted gravel berm in the ditch to provide the necessary support. A 3 ft. high gabion wall was constructed at the outside edge of the berm to form a catchment area for rock falls. No scaling of the slope was carried out because it was considered that the rock was so broken that scaling would only temporarily reduce the rock fall hazard.

This method of stabilization was adopted because the required support force at the toe was a modest value which could readily be achieved with the berm. Other alternatives, such as reducing the slope height or angle, would have been much more expensive because they would have involved excavation of large volumes of rock, and would also have disrupted traffic. Also, there was no opportunity for realignment of the slope because the track followed a river in the bottom of a steep sided canyon.

CONCLUSION

In conclusion, we consider that Goodman and Bray's limit equilibrium analysis is an appropriate engineering tool in the investigation and design of stabilization measures for toppling failures.

REFERENCES

[1] BARTON, N. (1971). A Model Study of the Behaviour of Steep, Excavated Rock Slopes, PhD. Thesis, Imperial College, Royal School of Mines, London.

[2] BURMAN, B.C. (1971). Some Aspects of the Mechanics of Slopes and Discontinuous Media, PhD. Thesis, James Cook University of N. Queensland, Townsville, Australia.

[3] BYRNE, R.J. (1974). Physical and Numerical Models in Rock and Soil Slope Stability, PhD. Thesis, James Cook University of N. Queensland, Townsville, Australia.

[4] CUNDALL, P. (1971). A Computer Model for Simulating Progressive Large Scale Movements in Blocky Rock Systems, Proceedings International Symposium on Rock Fractures, Nancy, France, Paper II-8.

[5] HITTINGER, M. (1978). Numerical Analysis of Toppling Failures in Jointed Rock, PhD. Thesis, University of California, Berkeley.

[6] GOODMAN, R.E. and BRAY, J. (1976). Toppling of Rock Slopes, Proceedings of Speciality Conference on Rock Engineering for Foundations and Slopes, Boulder, Colorado, Vol. 2.

[7] WYLLIE, D.C. (1980). Toppling Rock Slope Failures - Examples of Analysis and Stabilization. Rock Mechanics, 13, pp 89-98.

[8] HOEK, E. and BRAY, J. (1981). Rock Slope Engineering. 3rd Edition. Institute of Mining and Metallurgy, London.

ENGINEERING GEOLOGY OF VAIL PASS I-70

By

CHARLES S. ROBINSON1/ AND DALE M. COCHRAN2/

INTRODUCTION

Vail Pass is the divide between the West Fork of Ten Mile Creek on the south and Black Gore Creek on the north, on the west side of the Gore Range, in central Colorado (Figure 1). The section of Interstate Highway 70 across Vail Pass, which was the subject of this study, started at Wheeler Junction, on the West Fork of Ten Mile Creek, and crossed the pass down to approximately the junction of Gore Creek and Pitkin Creek. The geologic problems in the Vail Pass area in relation to the construction of I-70 were first recognized by R. K. Barrett, of the Colorado Division of Highways (Barrett, 1968). His report on the geology of the Vail Pass area was the basis for a contract with Charles S. Robinson & Associates. The field investigations were conducted by Dale M. Cochran and Charles S. Robinson, of the former firm of Charles S. Robinson & Associates, assisted by Gary T. Whitt, of R. V. Lord & Associates, in 1970-1971. Charles S. Robinson & Associates were retained in 1972-1973 by International Engineering, Inc. to assist in the environmental and design studies for I-70 on Vail Pass.

These engineering geologic studies of the Vail Pass area were the first major geologic studies conducted on a section of the Interstate Highway system in Colorado in advance of alignment selection and highway design.

1/ Mineral Systems, Inc., 5265 McIntyre, Golden, Colorado 80403

2/ Consulting Engineering Geologist, 2590 Cody Court, Lakewood, Colorado 80215

GENERAL GEOLOGY

Interstate Highway 70 across Vail Pass crosses terrain typical of the geology of the high mountains of Colorado (Figure 2). The pass is on the west flank of the Gore Range, a northwest-trending mountain range bordered on the east by the Blue River (Figure 1). The Gore Range is an uplifted block of Precambrian granitic and metasedimentary rocks flanked on the east and west by sedimentary rocks -- chiefly sandstone, siltstone, shale and some limestone. Surficial deposits, including colluvium, alluvium and glacial deposits formed or were deposited on the bedrock. As a result of glaciation and subsequent erosion, landslides developed in the bedrock and the surficial deposits, and were of major concern in the location of the highway alignment and design.

Previous geologic studies in the area were mostly general in nature and not directed specifically for engineering purposes. These studies included those by Lovering and Tweto (1944), on the Minturn Quadrangle, Bergendahl (1963, 1969), on the northern part of the Ten Mile Range and the southwest quarter of the Dillon Quadrangle, and Tweto and others (1970), on the Gore Range - Eagles Nest Primative Area. Reports on detailed geologic investigations by E. E. Walstrom and L. A. Warner, Consultants, and V. Q. Hornback, Geologist, Denver Board of Water Commissioners for proposed water collection and diversion systems on the west flank of the Gore Range in the vicinity of Miller and Black Gore Creeks, between Black Gore and Gore Creeks, and between Gore Creek and Booth Creek were made available to the authors.

Bedrock

The bedrock of the area of I-70 across Vail Pass includes Precambrian igneous and metasedimentary rocks upon which were deposited sedimentary rocks of Pennsylvanian and Permian age. These units were uplifted, folded and faulted in Pennsylvanian and Tertiary time. Figure 2 is a generalized geologic map of the Vail Pass area showing the distribution of the Precambrian igneous and metasedimentary rocks, and the major structural features. Figure 3 is a generalized stratigraphic column of the Vail Pass area.

The oldest rocks in the area crop out east of the highway in the central part of the Gore Range. They occur in road cuts along the highway north of Gore Creek, north of Vail Pass and west of Wheeler Junction at the south end of the Vail Pass section of the highway. They include Precambrian granitic and metasedimentary rocks and migmatite. The granitic rocks occur north, south and west of the junction of Gore Creek and Black Gore Creek (Figure 2). They include a gneissic granite -- the Cross Creek Granite (Bergendahl, 1969) -- which ranges in composition from a biotite-quartz diorite to a biotite granite. Inclusions of metasedimentary rocks in the gneissic granite are common. The inclusions are chiefly biotite - quartz - plagioclase gneiss and biotitic quartzite. The migmatite, west of Wheeler Junction, consists of alternating light and dark mineral layers, which range from less than an inch to several inches in thickness. The light-colored layers consist of plagioclase feldspar, quartz, microcline, and minor biotite. The dark-colored layers consist of plagioclase feldspar and biotite with minor quartz and microcline, and locally hornblende. The layers have been high folded and centered.

The sedimentary rocks include the Minturn Formation of Pennsylvanian age and the Maroon Formation of Pennsylvanian-Permian age (Tweto and others, 1970). The Jacque Mountain limestone (Figure 3) is considered as the top of the Minturn Formation. These formations were deposited on the Precambrian rocks of the Gore Range during the uplift of the range in Pennsylvanian-Permian times. The sedimentary rocks are exposed in road cuts along both lanes of the highway from about two miles west of Wheeler Junction to just south of Gore Creek (Figure 2).

The sedimentary rocks include sandstone, siltstone, shale and limestone. The sandstones are light pinkish gray to dark maroon and range from fine grained to coarse grained, and are locally conglomeratic. The siltstone and shale are reddish gray to dark reddish browh. The siltstone is micaceous, sandy and clayey. The shale is micaceous, sandy and silty. The limestone is restricted to the Minturn Formation and crops out only north of Miller Creek (Figure 2). The limestone is light pinkish gray to dark gray, and sandy and silty.

The Precambrian granitic and metasedimentary rocks were intensly folded and faulted in Precambian time and Tertiary time. The sedimentary rocks were folded and faulted during the Laramide Revolution and subsequently. The general structural features are shown on Figure 2. In general, the sedimentary rocks have been folded into a broad syncline that approximately parallels the highway. This syncline has been cut by a series of north to northeast trending faults. A major fault, or fault zone, the Gore fault, to the east of the highway, separates the sedimentary rocks from the Precambrian rocks of the core of the Gore Range.

Surficial Deposits

The surficial deposits include colluvium (including talus) derived from the weathering of the bedrock, alluvium (including terrace deposits and alluvial fans) deposited by running water, moraines deposited by glaciers and landslide deposits. Of these, the most important to the location and design of the highway were the landslides.

Colluvium, derived from the weathering of bedrock, occurs on both the Precambrian igneous and metasedimentary rocks, and on the Pennsylvanian-Permian sedimentary rocks. These deposits are generally thin (less than 5 feet in thickness) and in composition reflect the composition of the underlying bedrock. Alluvium, deposited by running water, reflects the character of the source of material.

The alluvium along the major streams and on terraces contains material derived from colluvium and the morainal deposits. The alluvial fans reflect the type of bedrock along the tributary streams. Major mountain glaciers moved down Gore Creek and West Ten Mile Creek into the valleys of Black Gore and Ten Mile Creeks. Morainal deposits, composed primarily of unsorted and unstratified material derived from Precambrian rocks, were deposited as the glaciers retreated. As the result of the over-steepening of slopes by glacial and subsequent stream erosion, failures occurred in the bedrock and the surficial deposits and landslide deposits were formed. Locally, swamp deposits have formed as a result of the damming of drainages by deposits of glacial material, landslides or by beavers.

ENGINEERING GEOLOGY

The engineering characteristics of the natural materials, and the occurrence of groundwater, determine the suitability of the material for foundation and for slope stability. The igneous and metasedimentary rocks are generally massive, competent rock. The upper meter or so, as a result of weathering, is generally rippable, but drilling and blasting were required in major cuts. Cut slopes in the foliated rocks stand vertically except where joints or fault systems approximately parallel the face of the cut. The excavated granitic and metasedimentary rocks served as excellent fill. Structures were founded on unaltered igneous and metasedimentary rocks.

The engineering characteristics of the sedimentary rocks are chiefly dependent upon the grain size, cementation, the percentage of clay, and the structure. Sandstone and conglomeratic sandstone constitute about 75 percent of the sedimentary bedrock. The other types of clastic sedimentary rocks are interbedded and gradational with the sandstone. The sedimentary rocks are generally massive, competent rock. The rock is generally rippable, but required drilling and blasting for the high rock cuts. The design of the cuts was controlled by variations in lithology and structure. In general, the beds dip gently and the joints were at right angles to the bedding. The attitude of the joints was used to determine alignment and slope of the cuts. The excavated sedimentary rock material served as excellent fill. Structures were founded on the massive sandstone and limestone beds. In placement of the fill on sedimentary rock, particular attention was paid to the direction of dip and groundwater drainage.

The surficial deposits of the area presented the major problems in the selection of the alignment and the design of the highway. The colluvial and morainal deposits were at their natural angle of repose and any steepening of the slope would result in raveling and slope failure. Construction of specially designed walls was used to control the cut slopes in these materials, and structures parallel to the slope were used to reduce the amount of cut and fill. The alluvial deposits presented a few problems. Most of the alluvial deposits in the major streams were bridged, or were thin enough that structures were founded on bedrock below the alluvium. The alluvial fans consisted of relatively well sorted and stratified granular material. Cut slopes of 1:1 were stable, if well drained. Some slopes were retained by specially designed walls.

The major geologic problem to the location and the design of Interstate Highway 70 on Vail Pass were the landslides. More than 50 percent of the alignment of the pre-existing U.S. Highway 6, north of Vail Pass, and 10 percent of the alignment south of Vail Pass had been constructed on landslides. Figure 4 is an index map showing the location of the major areas of landslides along the Vail Pass highway. It was recognized during the initial investigations that Interstate 70 could not be built across Vail Pass and avoid all the landslide areas. The initial geologic studies, and later more detailed studies, defined the landslides and their activity, and an alignment was chosen to avoid the more active areas. Following is a tabulation and brief description of the major landslide areas along the highway alignment (Figure 4).

Figure 4

Number	Approximate Station	Description
1	380 to 390	Stabilized landslide in igneous and metamorphic rocks (not outlined as active landslide area)
2	415 to 440	Bedrock slide of sedimentary rocks with cover of moraine and colluvium; two slides, one on either side of Black Gore Creek
3	480 to 520	Bedrock slide of sedimentary rocks with cover of rocky colluvium
4	540 to 560	Surficial slide in moraine and colluvium; older slide with active segments
5	610 to 675	Bedrock slide of sedimentary rocks with cover of moraine and colluvium
6	735 to 775	Bedrock slide of sedimentary rocks with cover of moraine and colluvium, very active locally
7	785 to 800	Bedrock slide of sedimentary rocks with cover of moraine
8	845 to 855	Bedrock slide of sedimentary rocks with cover of rocky colluvium on alignment of eastbound lane
		Copper Mountain slide, talus material, south of proposed alignments

The landslide areas include landslides in bedrock and in the surficial deposits.

Large areas of bedrock in the Vail Pass region have failed, forming landslide areas. The failure of the areas of bedrock is related to faulting or to oversteepening of the valley walls as a result of erosion related to glaciation. The landslides in bedrock have involved igneous, metasedimentary and sedimentary rocks.

An area of landsliding in the igneous and metamorphic rocks occurs south of the Gore Creek Campground. In this area, blocks of bedrock separated by arcuate faults form a series of steps from Gore Creek up the mountain to the south. The slope of the steps is toward the mountain to the south. Undrained depression or small draws developed at the back of the steps as a result of erosion along the fault or landslide slip plane. The faults or landslide slip planes were formed before glaciation since the outcrops show evidence of glaciation. The southern limit of the steps is a major northeast-trending fault. There is no evidence that there has been any recent movement of the landslide blocks, and the landslide is considered to be stable.

The largest landslides in the Vail Pass area involve bedrock of sedimentary rock. These landslides developed as a result of faulting and the erosion of Black Gore Creek. Most of the sedimentary rock landslides occur on the west or north side of Black Gore Creek north of Vail Pass. Only one landslide in sedimentary rock occurs south of Vail Pass. Most of the bedrock landslides are covered by surficial deposits. These surficial deposits are also moving locally because of movement in the bedrock or depositional instability. In such areas, landslides are sliding on landslides. Most of the areas of sedimentary rock landslides are stable; only locally within an area of a landslide are blocks of bedrock sliding.

The upslope limit of the landslide areas is typically a fault scarp. The faults are generally parallel to the valley and are believed to be related to the Gore Fault to the east of the area. The downslope toe of the landslides is Black Gore Creek.

Typical of the areas of sedimentary bedrock landslides are large, undrained depressions that resemble sinkholes in a karst region. These depressions generally have an elongated, elliptical shape. They range from less than 0.5 m (1.6 feet) wide and 2 m (6.5 feet) long to tens of meters wide and hundreds of meters long. Some depressions represent very old movement since they have trees as large as 0.3 m (1 foot) in diameter growing within their limits. Other landslides show, as evidence of recent movement, cracks in the sod in the bottom of the depression. Some of these cracks have been probed to depths of more than 5 m (16 feet).

Blocks of sedimentary rock are exposed in the landslide masses. Some of the blocks are tumbled, but most are about parallel in strike to the beds of the undisturbed bedrock, although the dip is greater. The blocks within a landslide area move different amounts at different times, as the differences in age of the depressions indicate. The blocks of sedimentary rocks are like randomly oriented and shaped wooden shingles on a steep roof; if one shingle is moved, support is removed from one or more other shingles and they also move, which in turn allows others to move until equilibrium is again established.

For the sedimentary rock landslides, Black Gore Creek erodes away a piece of sedimentary rock block, which allows the block to move toward the stream. Blocks upslope will then move downslope. This process has been going on over a long period of geologic time. Because these landslides are composed of blocks of sedimentary rock, and the movement is generally along and about parallel to the bedding, the landslides are not thick. Drilling and the installation of instruments have indicated that the maximum depth of movement in this type of slide ranges from 6 to about 21 m (20-70 feet). Movement does not occur along a single slip plane; rather, there may be several parallel slip planes in a single area of landslide. Movement occurs along weak bedding planes between beds in a single block of sedimentary rock or between overlapping landslide blocks of sedimentary rock.

Landslides have developed in the surficial deposits on both sides of Vail Pass. The landslides are the result of the deposition or accumulation of surficial deposits on steep bedrock slopes and then the periodic movement of the deposits. Failure has been the result of steepening of the slopes by erosion, by changes in the groundwater regime, or by movement of the bedrock. All types of surficial deposits are involved in landslides, but most of the landslides are found in the moraine or colluvium.

The scarps of the individual slumped areas are recent. Most indicate movement of the surficial deposits during historic times; many show annual movement. The individual scarps range in length from 1 to about 150 m (3.3-500 feet). They average about 60 m (200 feet) in length. The vertical offset across the scarps ranges from less than a meter to several meters. The abundance of scarps in an area of landslides and their relative short length and small offset indicate that the depth of movement is relatively shallow. Drilling has shown that the individual landslides in the surficial deposits are generally less than 15 m (50 feet) thick.

The individual scarps may represent a single small landslide or an active segment of a large landslide area. The landslide area may be (a) a bedrock landslide in which the scarp represents slumping in the surficial material as a result of movement in the bedrock or (b) a large landslide in the surficial deposits. The individual scarps indicate the active segment of the large landslide.

A landslide in the surficial deposits is the result of the deposition of these deposits on a bedrock slope that was oversteepened by glacial erosion. The deposits were poorly drained at the time of deposition, and many swamps developed. The permeability of the surficial deposits is different in different areas, and numerous springs occur throughout. During much of the year when the surface is not frozen, the deposits are saturated with water. With the spring runoff there is a period of rapid erosion during the time of maximum saturation. This results in minimum stability in a mass of poorly sorted and unconsolidated material on a steep slope, the lower edge of which is at a stream channel. Along Black Gore Creek, downcutting of the creek has not been able to keep up with the movement of the landslides into the valley. The gradient of the creek above the landslides has been lowered and alluvial deposits have formed because the landslides have partially dammed the creek.

ENGINEERING SOLUTIONS, GEOLOGIC PROBLEMS

The geologic problems were defined by intermediate and detailed geologic investigations. These data were integrated with the standards for Interstate Highway construction and a preliminary alignment and design were established.

Slope stability was maintained by careful design of back slopes. The eastbound and westbound lanes were separated where space, line and grade allowed, and the heights of back slopes and fills were kept to a minimum by construction of specially designed walls. Slopes in surficial material were laid back as far as was practical, contoured, covered with topsoil, and seeded. Particular care was taken to control surface and groundwater drainage on all cut-and-fill slopes. On bedrock slopes, the different types of material were treated differently depending on their ability to stand. The slopes in more competent units approached vertical, whereas those in less competent units were laid back and often seeded. The natural breakage of the rock, e.g., along joints, was followed, and the back slopes in bedrock conform in appearance to natural rock slopes.

One area of slope stability that was of particular concern was that between Gore and Bighorn Creeks. Along the alignment were surficial deposits, mostly glacial moraine, on Precambrian igneous rock. The surficial deposits were at their maximum angle of repose and were saturated with groundwater during most of the year. The choices for the placement of the highway were restricted by the presence of privately owned lands in the valley. Cuts in this area could have caused major slope failure. The solution was to put much of the highway on a structure.

It was not possible to avoid all landslide areas and still maintain highway standards. The studies, including the instrumentation, had indicated the landslide areas that were the most active and those that were relatively stable. The highway was designed to avoid the more active landslides. Where the highway crossed older landslides, the lanes were separated as far as possible in conformity with the requirements of standards for line and grade. The separation of the lanes allowed the height of cuts and fills to be held to a minimum. In several landslide areas, groundwater drainage was increased by drilling horizontal drains well below the level of the highway and the fills. The drainage across the old landslides was improved and carefully controlled to reduce infiltration into the landslide mass.

One landslide area was located between stations 415 and 425. Active bedrock landslides from either side of Black Gore Creek had their toes in the creek. The landslides on either side of the creek were instrumented, and their movement was monitored. The maximum movement occurred during the period of high groundwater levels, i.e., spring runoff. The solution to the stabilization of these two landslides was to fill the valley, transferring the thrust of one landslide against the other and putting the stream and the highway on the valley fill.

REFERENCES CITED

Barrett, R. K., 1968. Preliminary Geologic Report: Gore Creek Campground to Wheeler Junction. District 3, Colorado Department of Highways, 20 pp.

Bergendahl, M. H., 1963. Geology of the Northern Part of the Tenmile Range, Summit County, Colorado. U. S. Geological Survey, Bull. 1162-D, 19 pp, 1969.

Bergendahl, M. H., 1969. Geologic Map and Sections of the Southwest Quarter of the Dillon Quadrangle, Eagle and Summit Counties, Colorado. U. S. Geological Survey, Miscellaneous Geological Investigations Map I-563.

T. S. Lovering and O. Tweto, 1944. Preliminary Report on Geology and Ore Deposits of the Minturn Quadrangle, Colorado. U. S. Geological Survey, openfile report, 115 pp., map.

O. Tweto, B. Bryant, and F. E. Williams, 1970. Mineral Resources of the Gore Range-Eagles Nest Primitive Area, Eagle and Summit Counties, Colorado. U. S. Geological Survey, Bull. 1319-C, 127 pp., map.

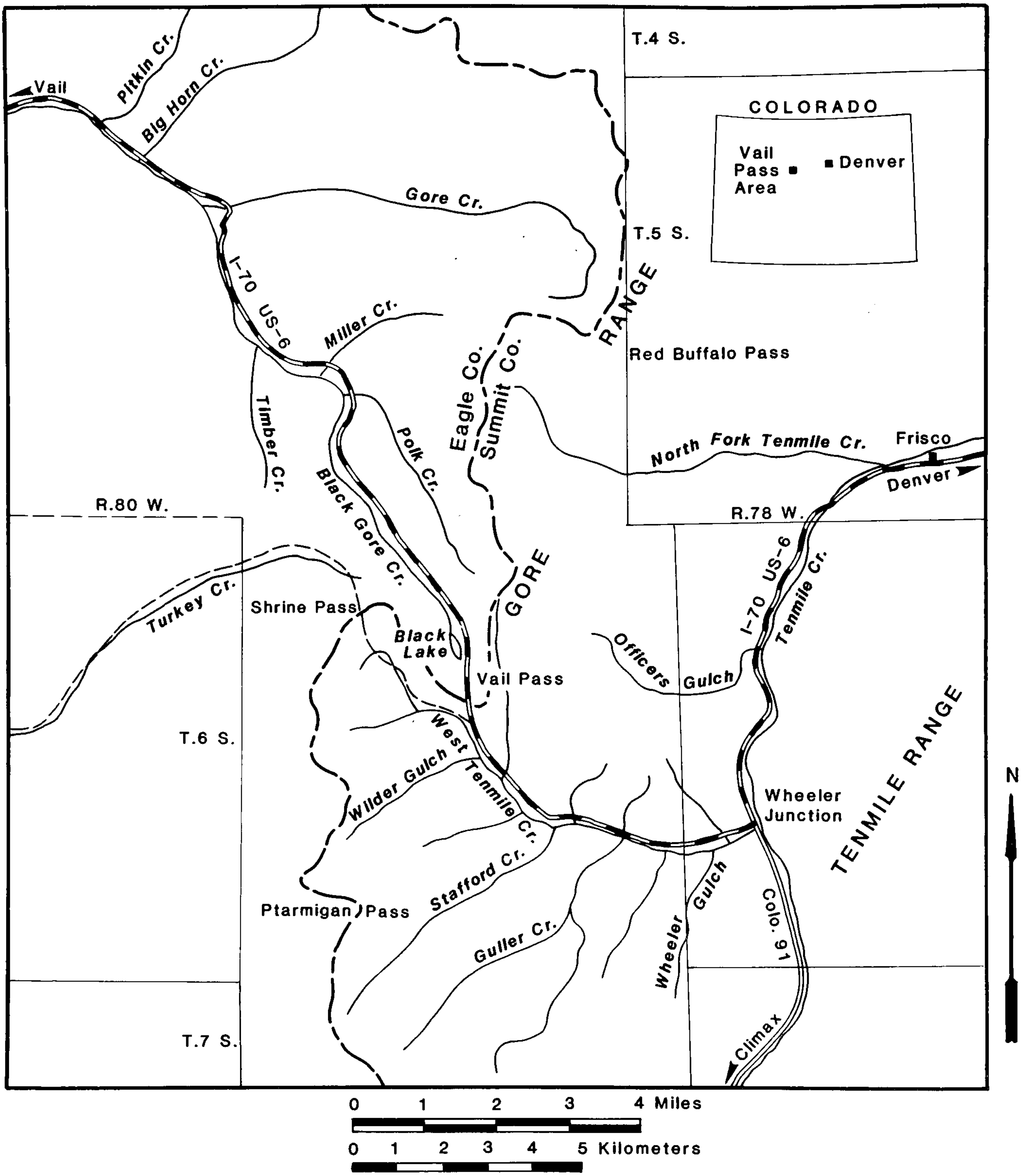


Figure 1.--INDEX MAP, INTERSTATE HIGHWAY 70, VAIL PASS, COLORADO

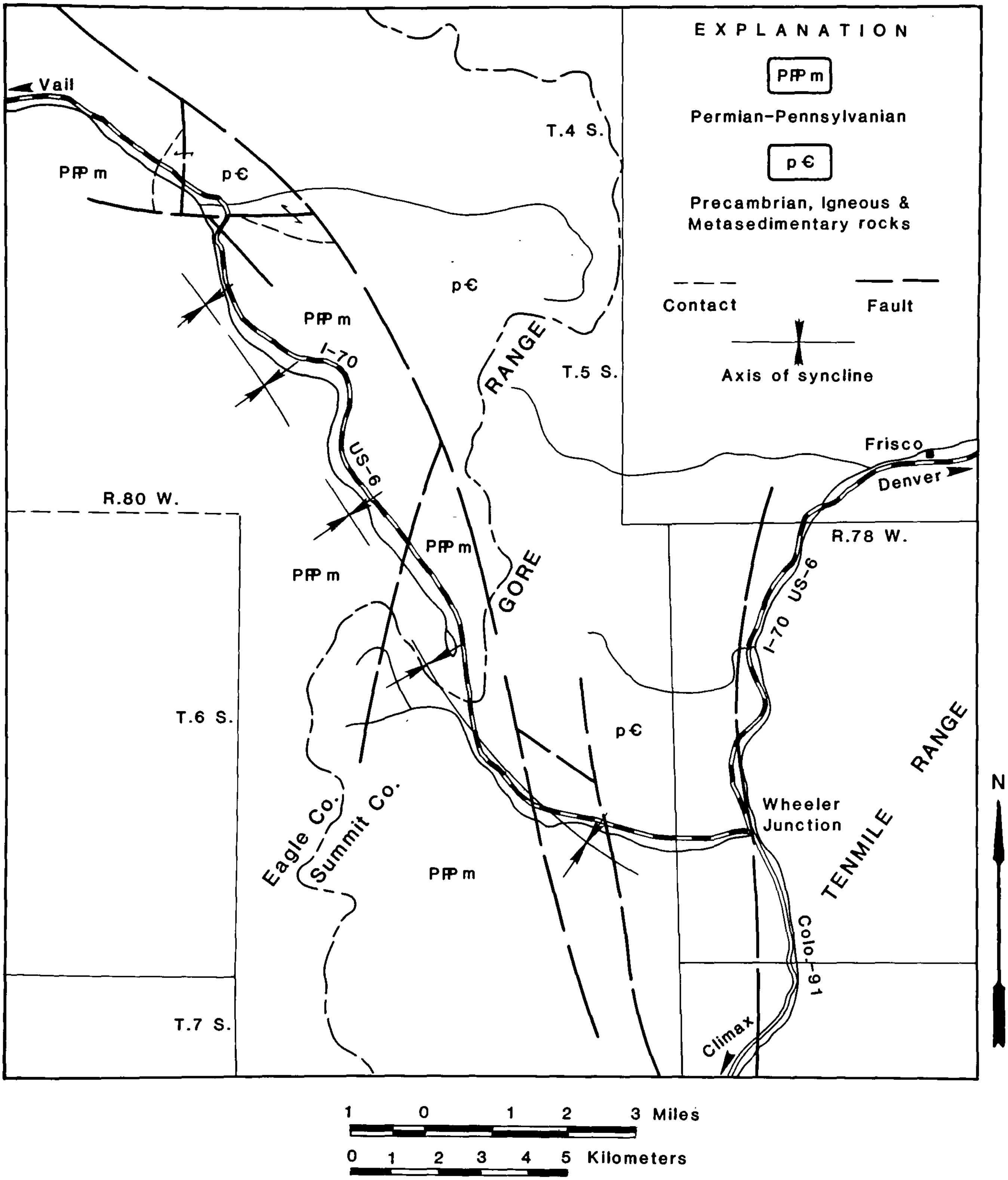


Figure 2.--GENERALIZED BEDROCK GEOLOGIC MAP, VAIL PASS AREA, COLORADO

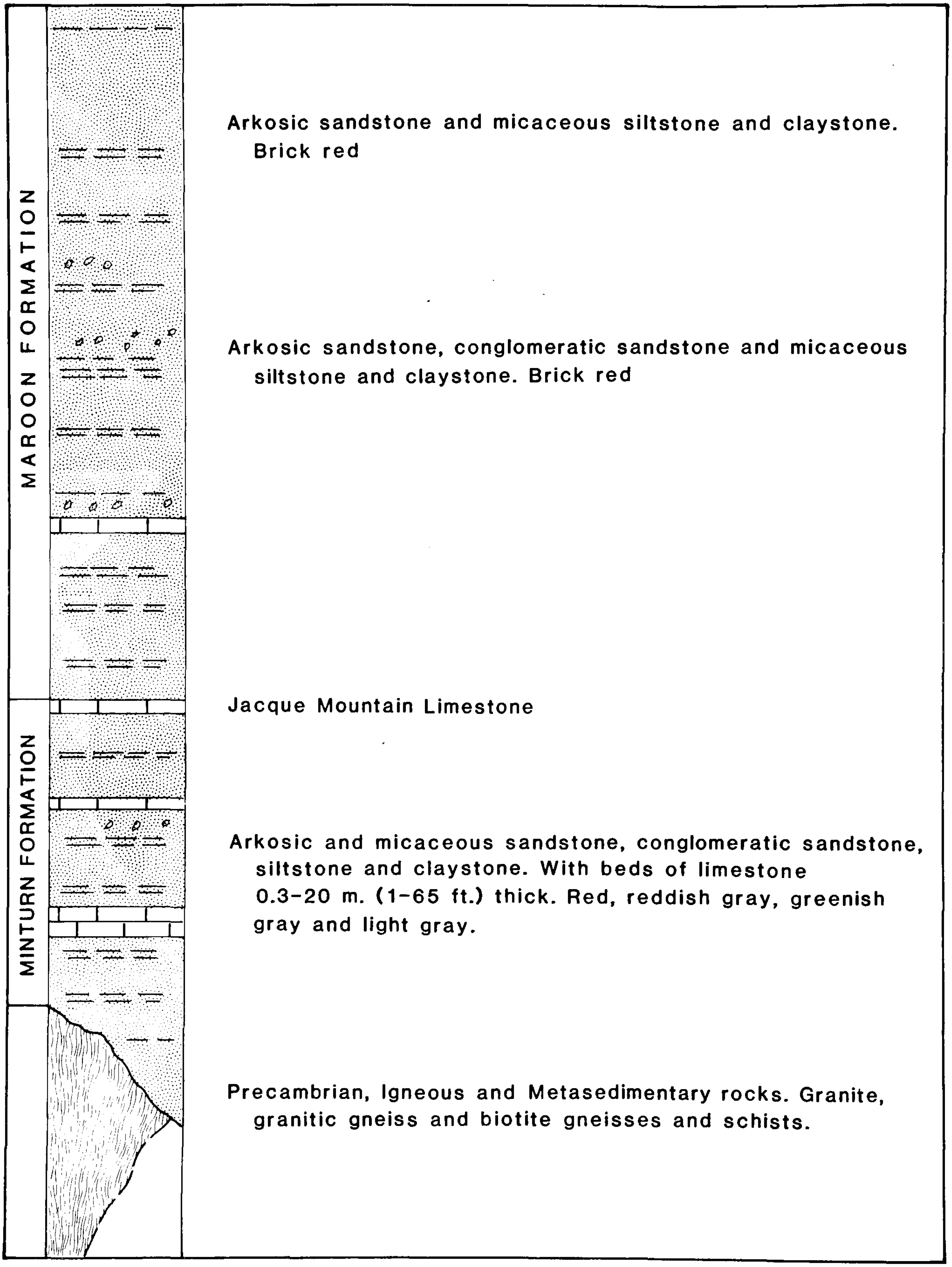


Figure 3.--GENERALIZED STRATIGRAPHIC COLUMN, VAIL PASS AREA, COLORADO

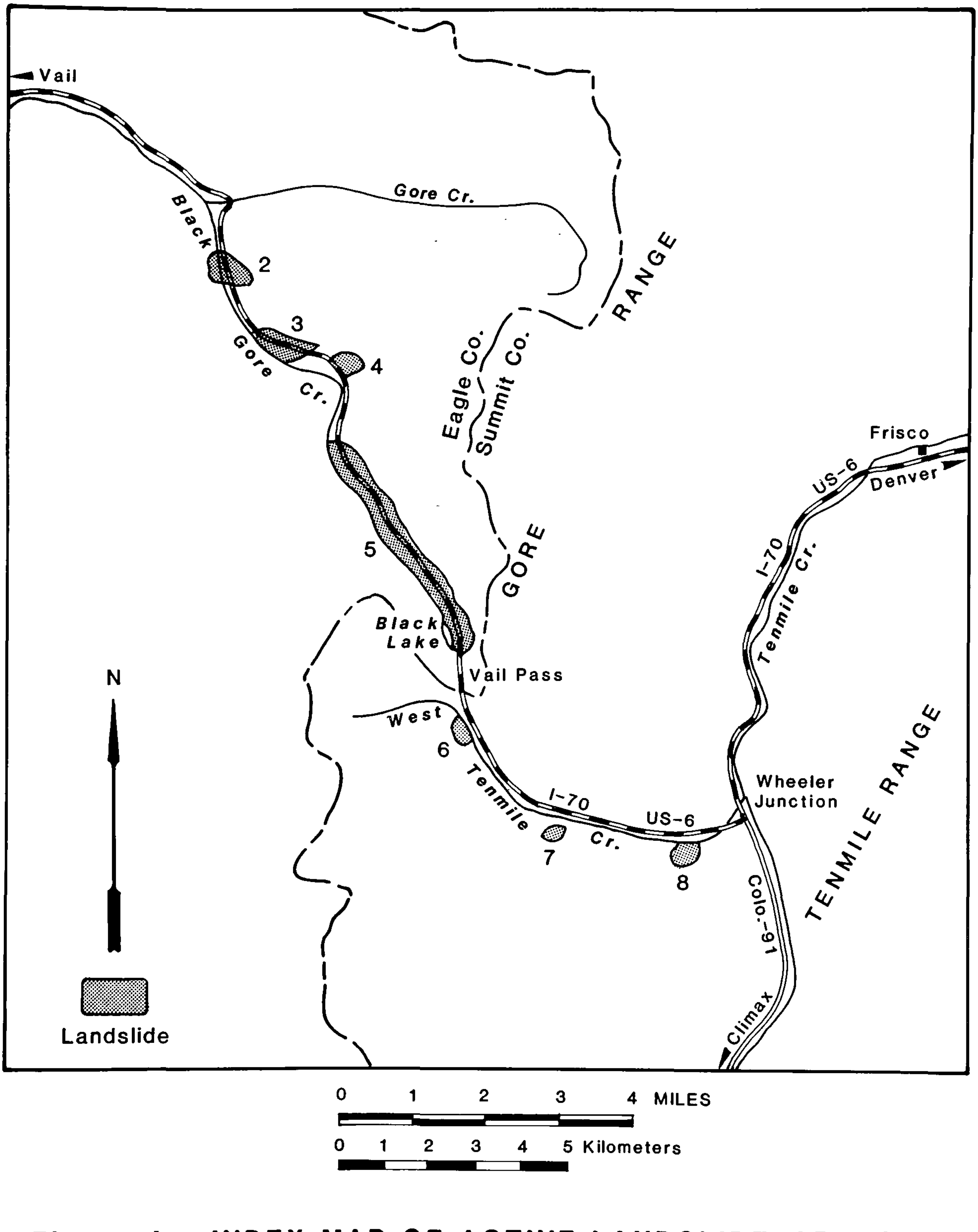


Figure 4.--INDEX MAP OF ACTIVE LANDSLIDE AREAS, VAIL PASS AREA, COLORADO

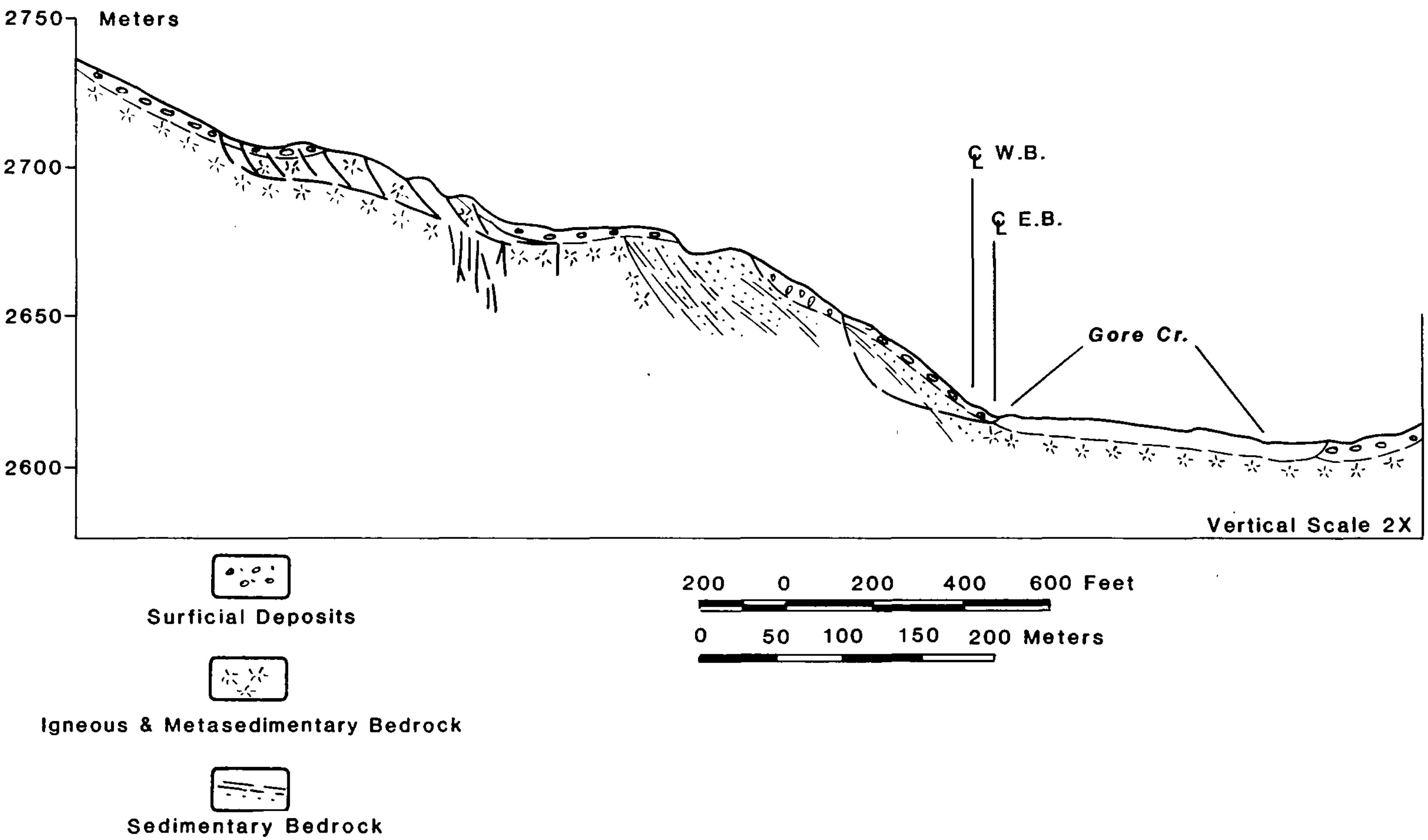


Figure 5.--CROSS SECTION OF BEDROCK LANDSLIDE IN IGNEOUS AND METASEDIMENTARY ROCK (Station 3)

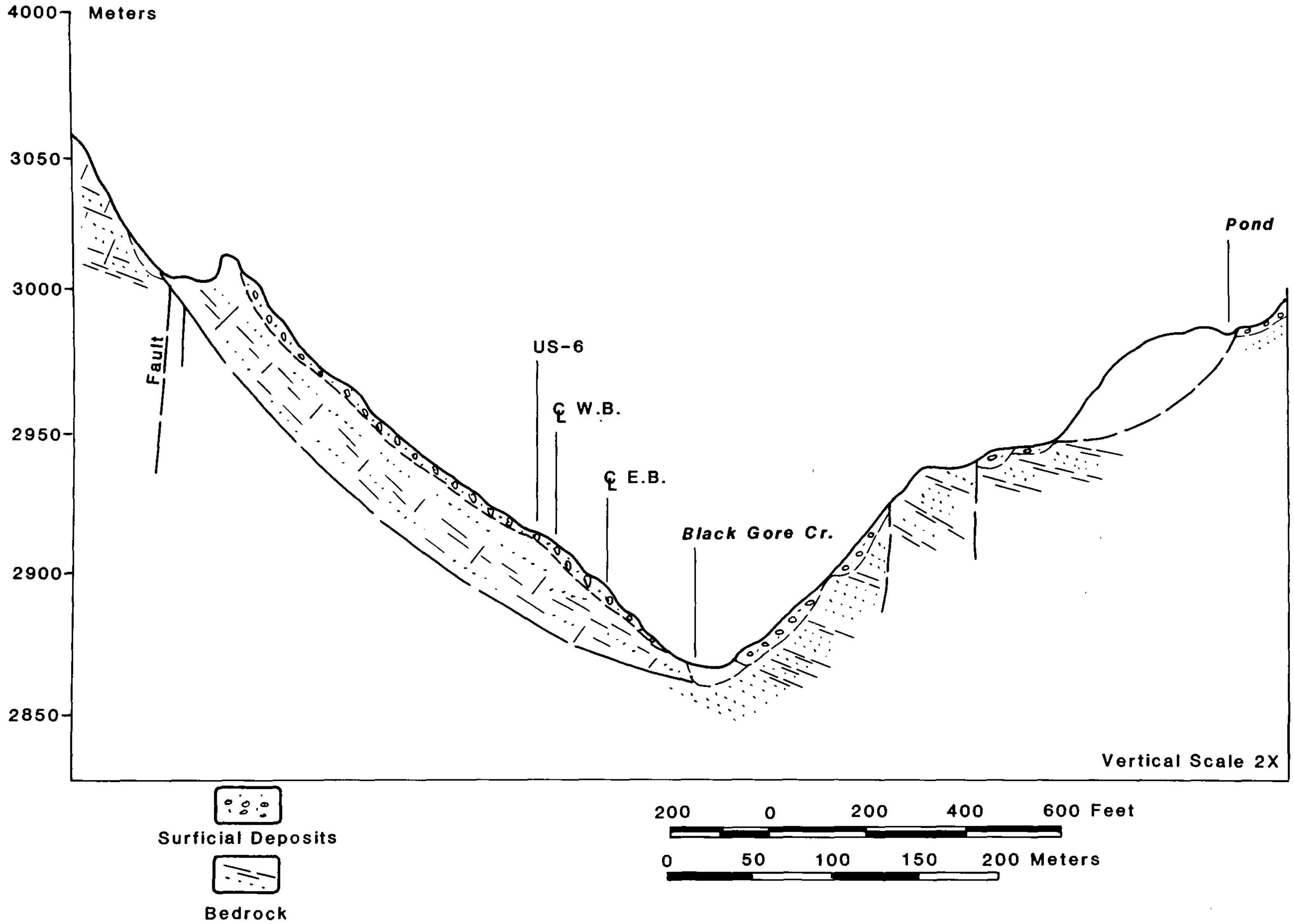


Figure 6.--CROSS SECTION OF BEDROCK LANDSLIDE IN SEDIMENTARY ROCK VAIL PASS AREA, COLORADO

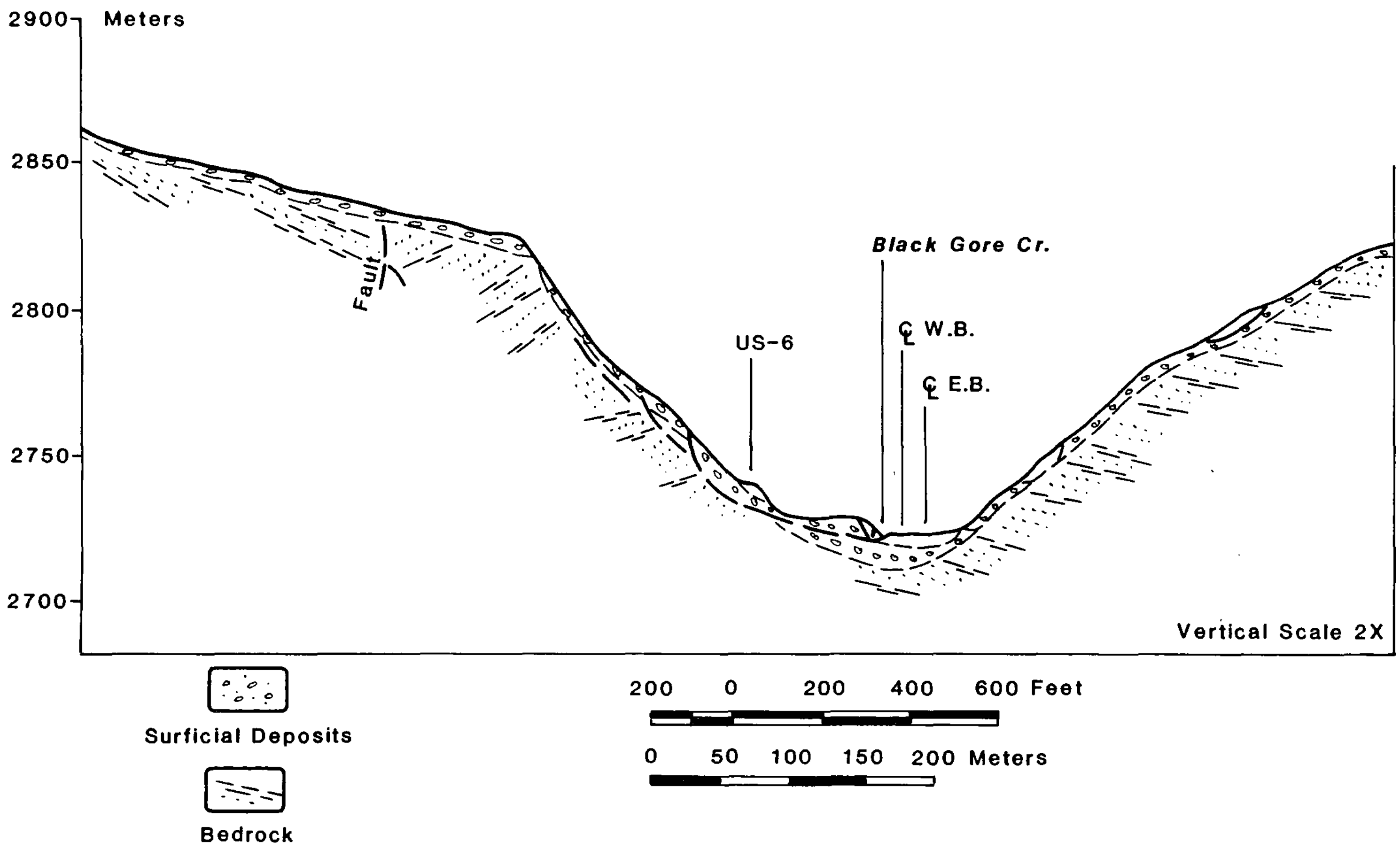


Figure 7.--CROSS SECTION OF LANDSLIDE IN SURFICIAL DEPOSITS VAIL PASS AREA, COLORADO

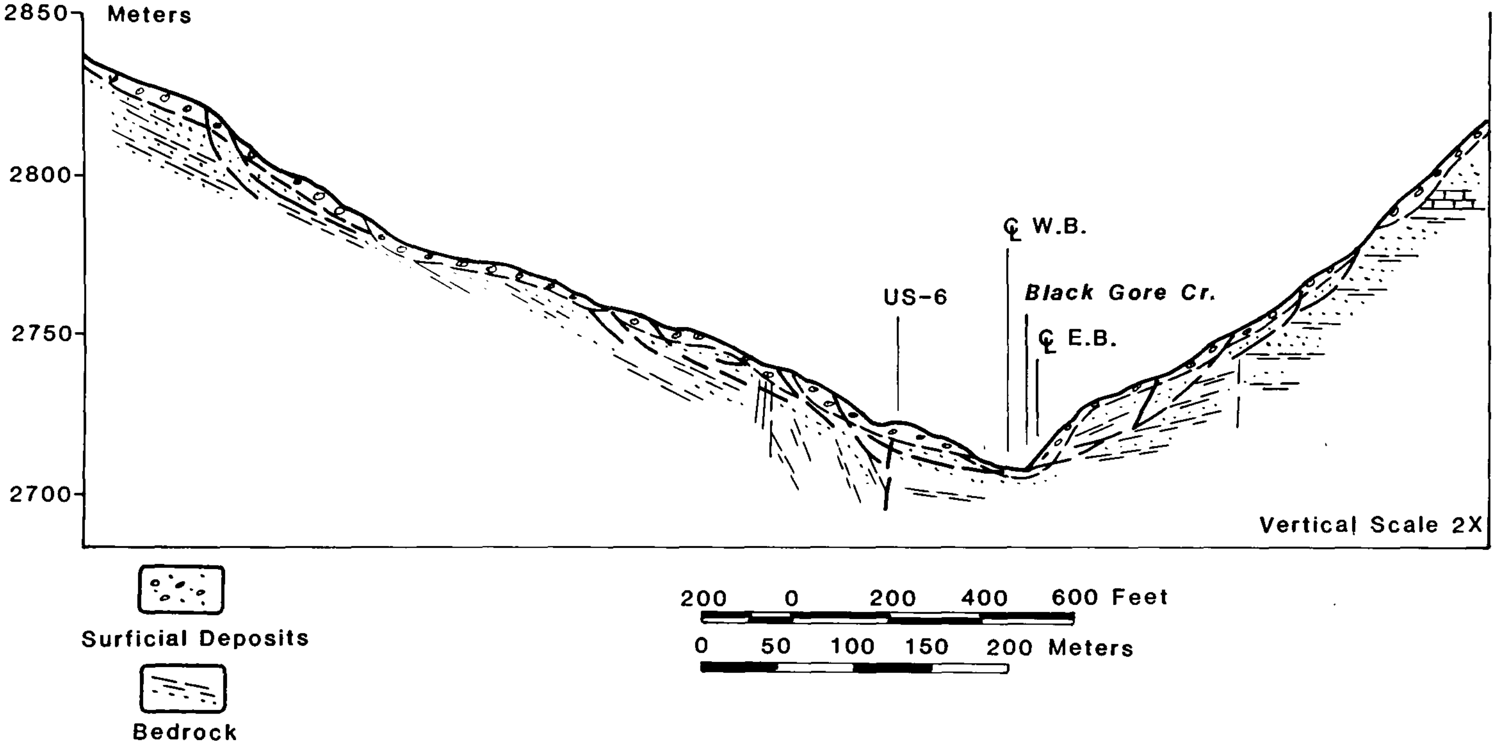


Figure 8.--CROSS SECTION THROUGH ABUTING LANDSLIDES

VAIL PASS AREA, COLORADO

GEOLOGY OF THE GLENWOOD CANYON ALONG I-70

by

Jeffrey L. Hynes

Senior Engineering Geologist

Colorado Geological Survey

Denver, Colorado

This paper describes the general geologic environment of the Glenwood Canyon. Stratigraphy, structure and engineering characteristics are presented as a preview to the second half of the field trip program.

Much of the information presented here is based upon the earlier work done in the canyon by Bass & Northrup, Gardner and McQuown and the Colorado Department of Highways.

Physiographic Setting

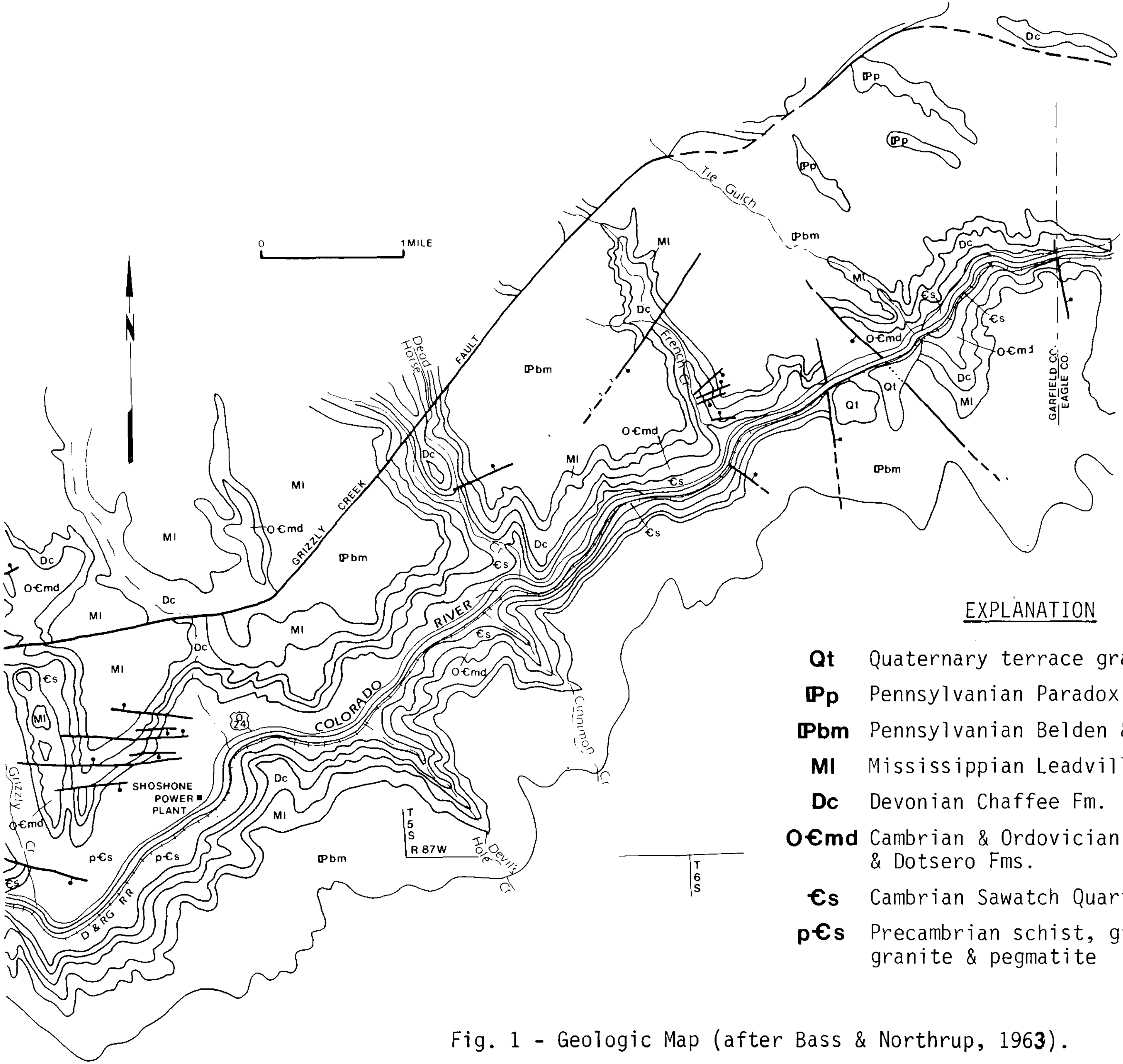
The Glenwood Canyon is deeply incised through several thousand feet of early Paleozoic sedimentary and pre-Cambrian igneous and metamorphic rocks of the White River Uplift (Fig. 1). The bottom of the canyon, also known as the Narrows, contains Quaternary deposits including talus, debris fans, terrace deposits and thick clay beds (Fig. 9).

The steep canyon walls support little vegetation and most of that is restricted to evergreens on the various benches produced by the more resistant units. The river banks and side streams are generally well vegetated with willows and a brushy understory.

Stratigraphy (Figs. 1, 2 & 3)

Igneous and metamorphic rocks of pre-Cambrian age are exposed in the lower portions of the canyon from French Creek westward to No Name Creek. These rocks consist primarily of quartz-biotite schists and greenstones with granitic and pegmatitic intrusions. The greatest majority of these intrusions are intercepted by the relatively uniform erosional unconformity which separates the unit from the Paleozoic sediments above. Maximum exposure of this rock type is greater than 700 feet in the general vicinity of the Shoshone power plant.

Overlying the pre-Cambrian unconformity is the Sawatch Quartzite. This unit is composed of 500 feet of bedded sandstones, quartzites with thin beds of glauconitic shale and dolomite at the top.



EXPLANATION

Qt	Quaternary terrace gravels
ℙp	Pennsylvanian Paradox Fm.
ℙbm	Pennsylvanian Belden & Molas Fms.
Ml	Mississippian Leadville Limestone
Dc	Devonian Chaffee Fm.
O€md	Cambrian & Ordovician Manitou & Dotsero Fms.
€s	Cambrian Sawatch Quartzite
p€s	Precambrian schist, greenstone, granite & pegmatite

Fig. 1 - Geologic Map (after Bass & Northrup, 1963).

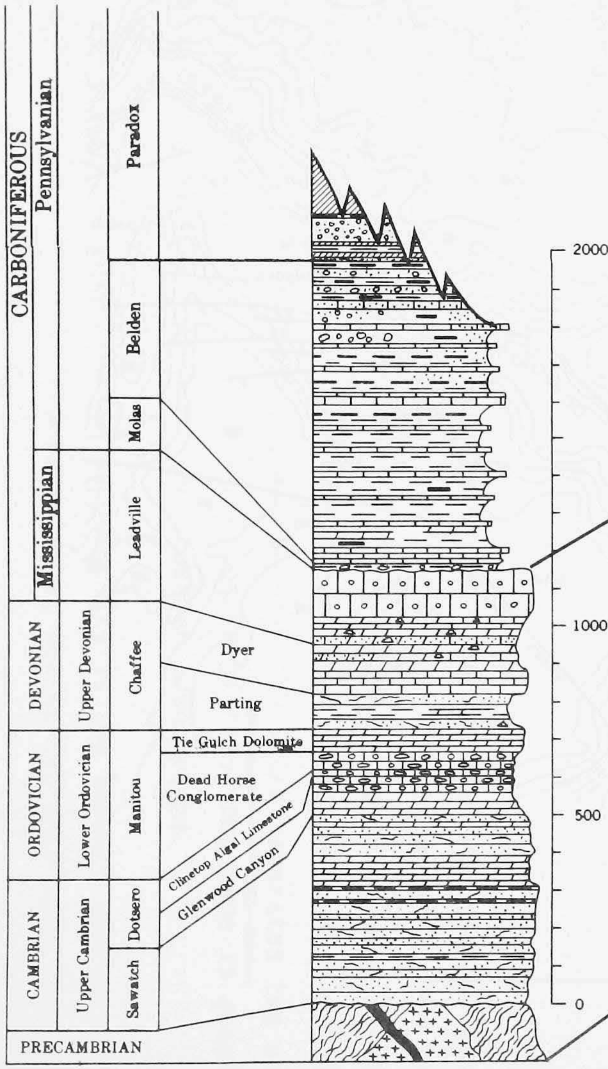


Fig. 2 - Stratigraphic Column (from Bass & Northrup, 1963).

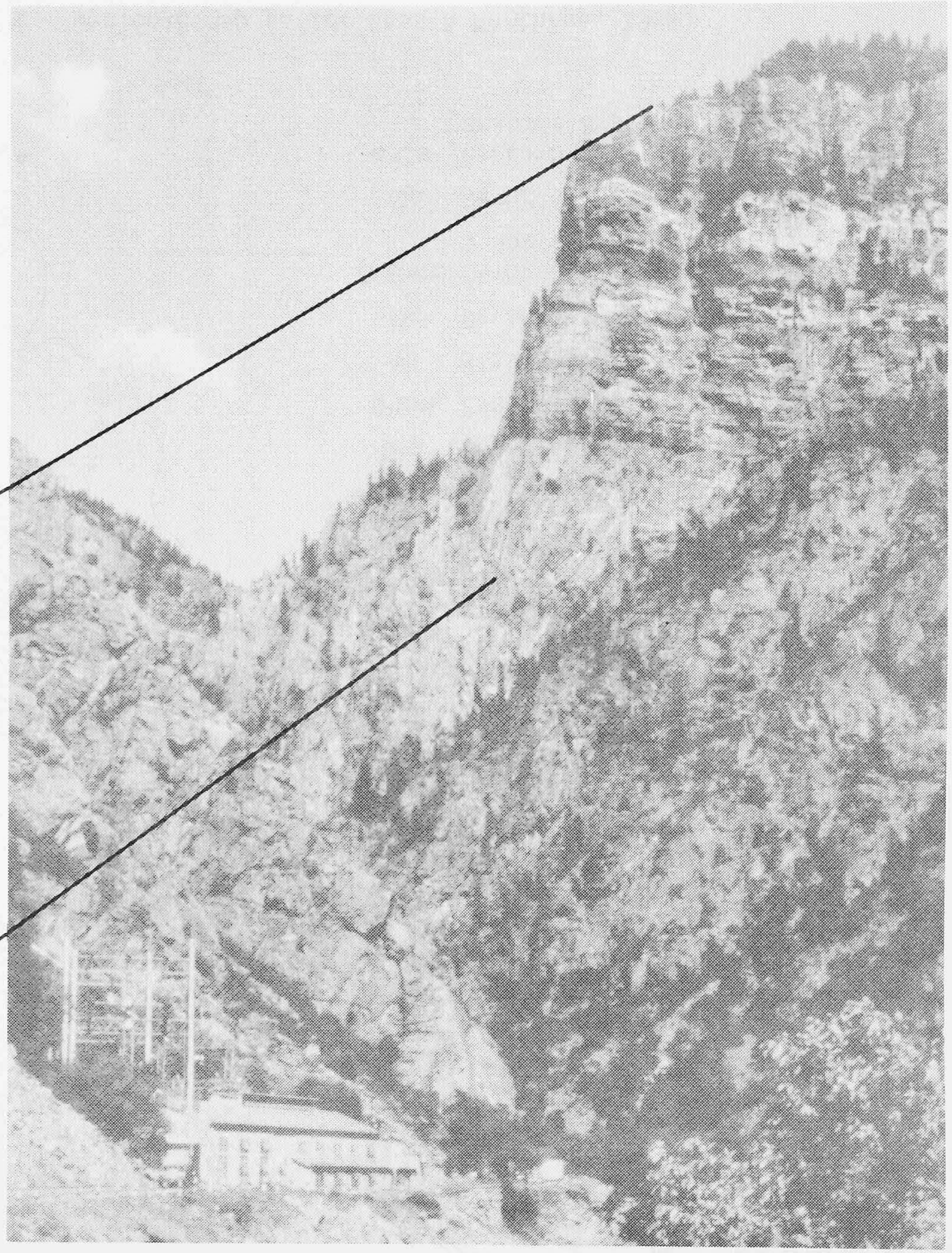


Fig. 3 - Outcrop in south canyon wall by Shoshone Power Plant.

The Dotsero Formation, late Cambrian age, which conformably overlies the Sawatch consists of about 100 feet of thin bedded dolomites and limestone pebble conglomerates interbedded with thin dolomitic shales. The upper beds of the Dotsero represent one of the best marker units in the canyon. They consist of a ledge forming algal limestone which weathers to a very light grey or white and can be readily recognized throughout the area. (Fig. 4).

Above the Dotsero is the Manitou Formation of early Ordovician age. It is made up of 150 feet of thin, bedded limestones, limestone conglomerates and thin micaceous shales grading to dolomites in the upper 50 feet, Tie Gulch Member. This dolomite sequence usually forms a prominent bench above which the less resistant Chaffee Formation has eroded back leaving a residual soil which supports substantial vegetation.

The late Devonian Chaffee Formation, unconformably overlies the Tie Gulch but the bedding in the two is essentially conformable. The Chaffee is nominally 250 feet thick and is composed predominantly of shales, sandstones and dolomites in the lower portions (Parting Member) to thick, clean, fossiliferous limestones and dense dolomites in the upper part (Dyer Member).

Above the Chaffee Formation is the Leadville Limestone of Mississipian age. This unit is composed of 200 feet of predominantly massive oolitic limestone but tends to sandy dolomite and dolomitic sandstones at the base.

In most locations the Leadville Limestone caps the cliffs on both sides of the river and in some exposures shows a marked karst-like upper unconformity (Fig. 5).

Within the actual Narrows, little of the Pennsylvanian Molas and Belden shales have been preserved. These units can be observed to the east of the canyon as one progresses upsection into the Paradox Formation where the physiography abruptly changes to badlands in the vicinity of the confluence of the Colorado and Eagle Rivers.

Quaternary deposits of talus, debris fans, alluvium and clay lenses can be observed in a narrow, discontinuous zone bounded by the river channel and the confining bedrock cliffs (Fig. 9). These deposits generally underly the actual roadway alignment and their presence and abrupt boundaries represent the major engineering geological implications associated with the construction of the highway.

Just to the north of the canyon are glacial deposits, volcanic ash beds, and travertine zones but none of these directly affect the project.

Fig. 4 - Marker bed (Clinetop Member) accentuating horst-and-graben structures just each of Grizzly Creek.

Fig. 5 - Relic karst terrane and solution features at the top of the Leadville Limestone just north of Golden Bear Ranch.

Structure

The general structure of the White River Uplift is a large dome with over 10,000 feet of relief (Fig. 6). The dome is slightly elongate along a northwest trend roughly parallel to the axis of the Piceance Basin to the west. It is the intersection of this axis with the incised river channel which exposes the lower Paleozoic section and crystalline basement in this area.

The major structural feature in the area is the Grizzly Creek Fault, a large thrust fault with as much as 1,000 feet of throw, which roughly parallels the river channel approximately 1.5 to 2 miles to the north (Fig. 1).

Associated with this faulting are numerous swarms of small throw (generally less than 50 feet) faults producing horst and graben structures in the sedimentary sequences exposed in the high cliffs on the north side of the Colorado River (Fig. 4). This faulting does not appear to penetrate very far below the top of the crystalline basement and in general produces only secondary problems associated with the highway. The most notable one being the direct correlation between large, coarse talus slopes and the fault zones.

Normal to the trend of these structural features is a graben which trends N30^{O}W, has on the order of 500 feet of relief and is about a half-mile wide along the road alignment (Fig. 1). The origin and relationship of this feature are not well understood but it may be associated with some rift-like responses along the SE margin of the White River Uplift.

Jointing and fracturing of the brittle sedimentary sequence is extensive. The crystalline complex appears to be less so. Among the most pronounced visible evidence are the large, joint controlled, subvertial canyons developed in the Sawatch Quartzite at the eastern end of the Narrows (Fig. 7). Measurements taken by MacQuown indicate a conjugate set of vertical joints trending N15^{O}E and N80^{O}W. This combined with the subhorizontal bedding produces a nearly orthogonal pattern with resultant subcubic talus blocks.

Highway Impacts

The inplace bedrock, where exposed along the right-of-way, possesses excellent bearing and stability characteristics. If there is a problem associated with the bedrock it is most certainly the significant risk of rockfall associated with almost the entire length of the Narrows. (Fig. 8).

The principal engineering problems posed by the unconsolidated materials are the stabilization of talus slopes and the accomodation of the mountain torrent type

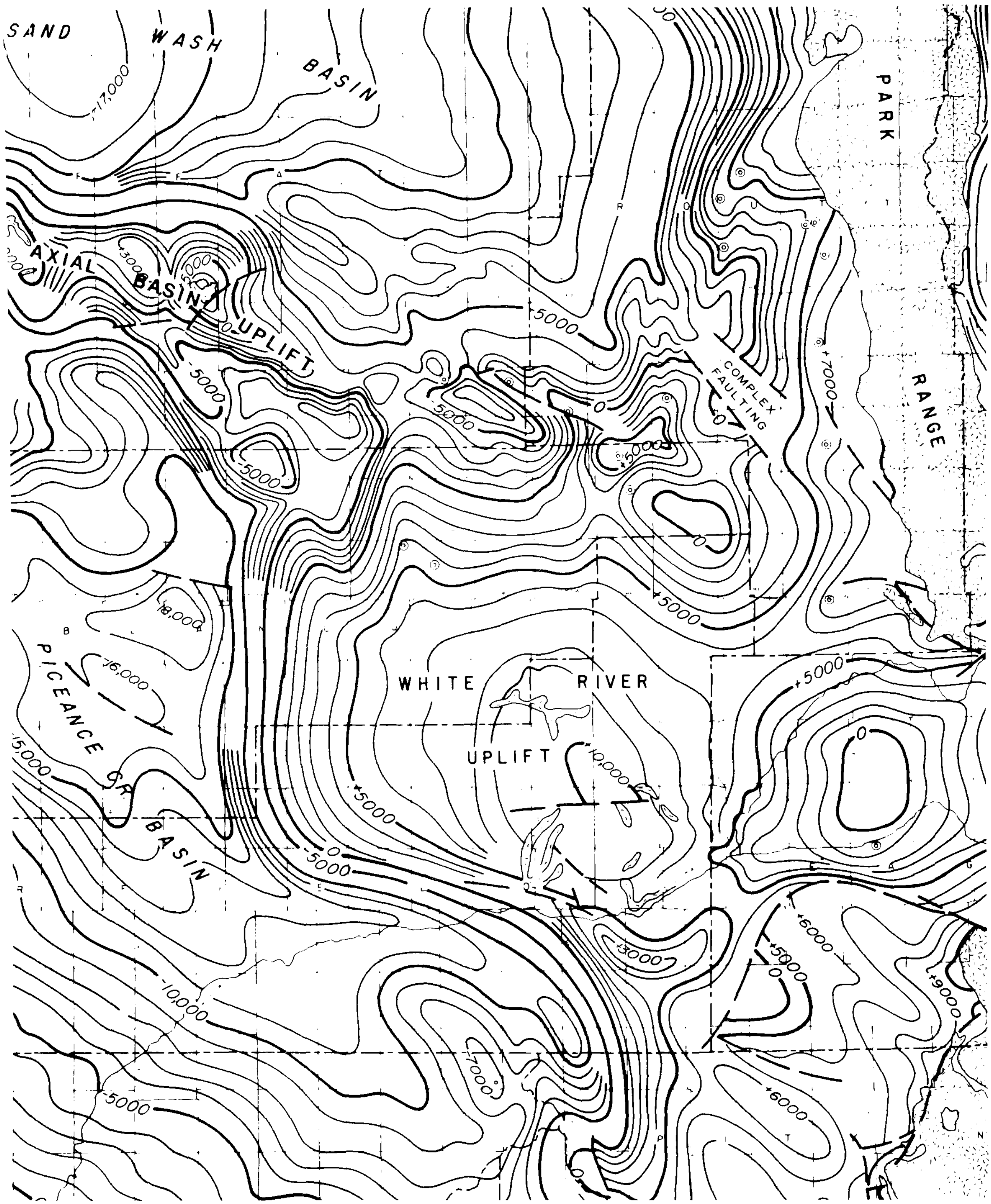


Fig. 6 - Structure contours on the top of the crystalline complex (from MacLachlan, 1969).

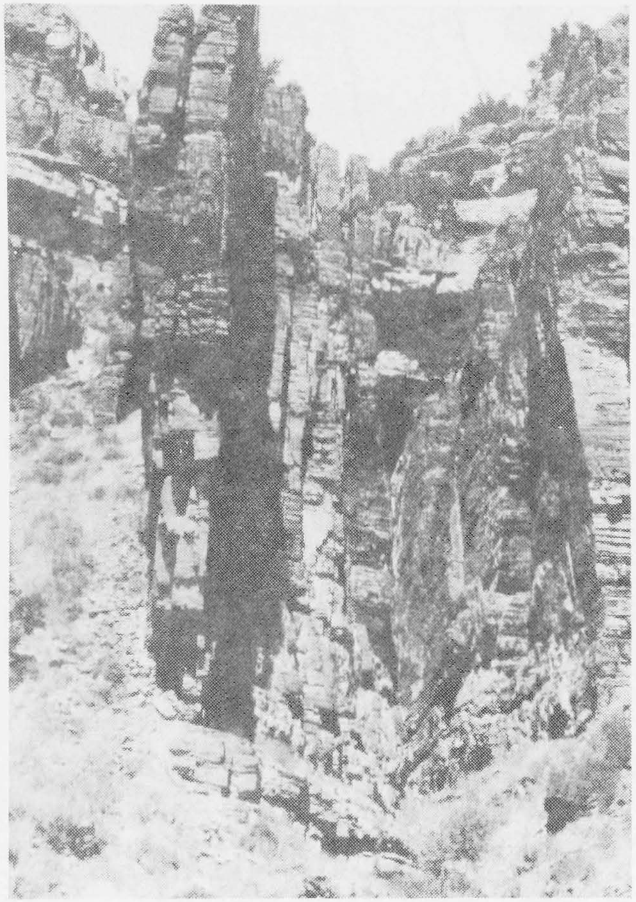


Fig. 7 - Massively jointed Sawatch Quartzite forms Book Cliffs near eastern end of the Narrows

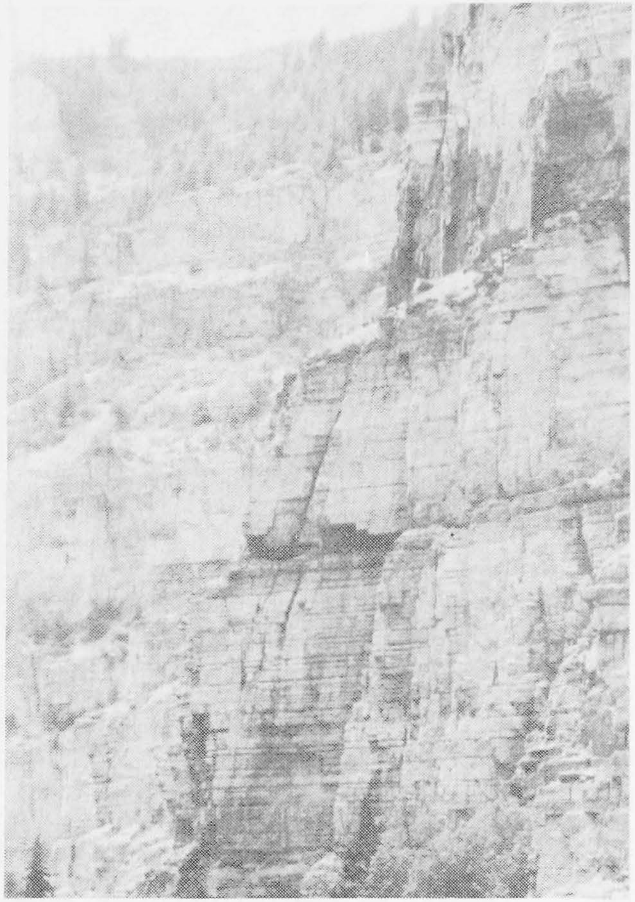


Fig. 8 - Rockfall hazard near Deadhorse Creek, note the continuity of jointing and "wedging" of lower block.

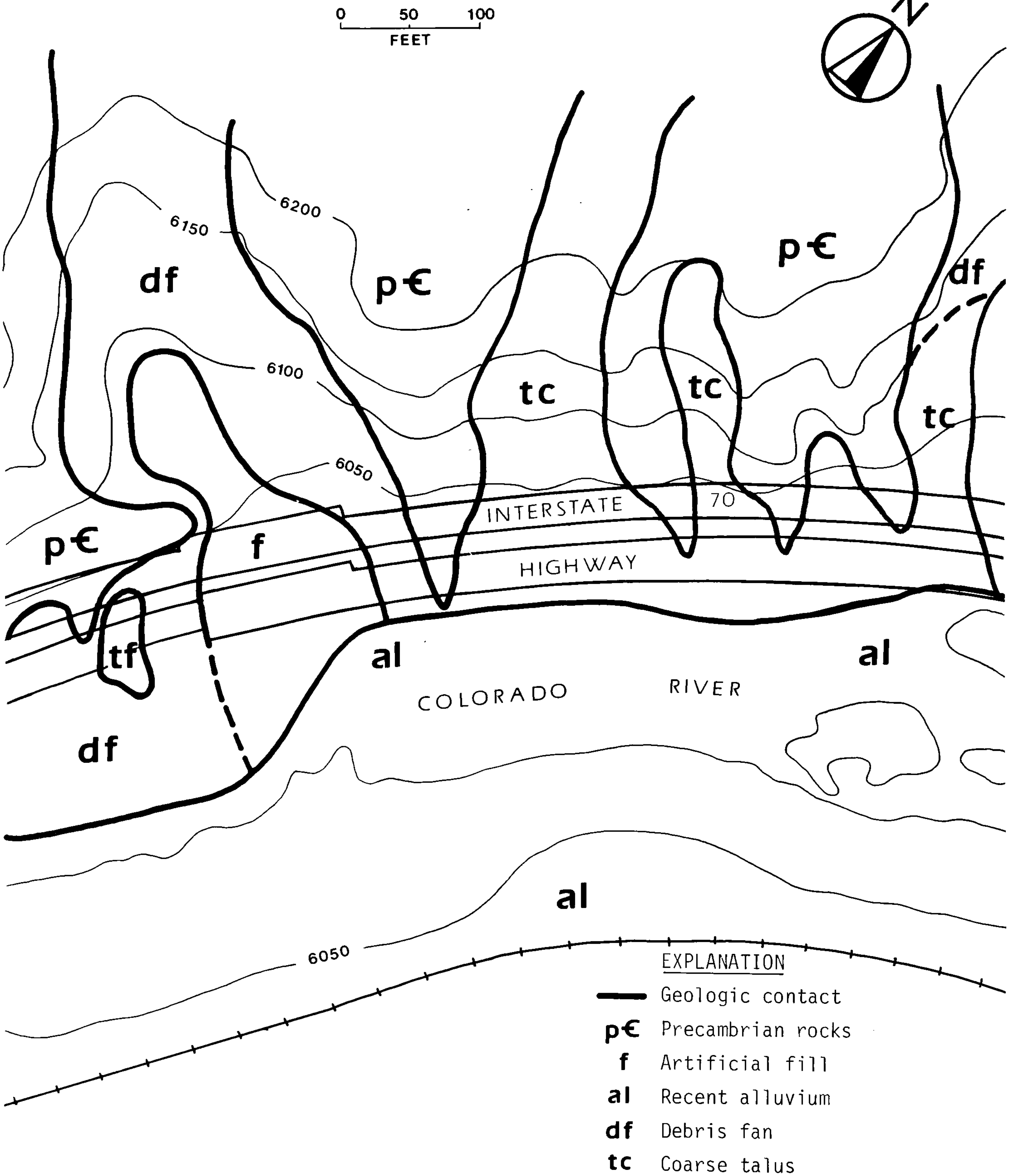


Fig. 9 - Engineering geology and surficial deposits along a reach of I-70 just west of Hanging Lake Reststop (from Colorado Dept. of Highways).

of drainage associated with the side streams and their debris fans. The engineering solutions to some of these problems will be the focus of parts of tomorrow's field trip. (Figs. 10 and 11).

Additionally, several very thick clay deposits have been discovered during the exploratory boring program. These deposits appear to be associated with lake formation along various reaches of the river due to rockfall, debris flooding, periglacial activity or perhaps even fault movement. The origin of these deposits is still unknown. The problem associated with them is one of very low bearing strength and excessive depth to firm bedrock below. Another paper will be presented to show how one solution to this problem is working, and the site will be visited tomorrow.

References

Bass, N.W., and Northrup, S.A., 1963. Geology of the Glenwood Springs Quadrangle and Vicinity Northwest Colorado, USGS Bull. 1142-J.

Colorado Department of Highways and Federal Highway Administration, 1978. Glenwood Canyon I-70 Design Report, Vols. I and II.

Gardner, M.E., . Unpublished maps of the Glenwood Springs Canyon Quadrangles.

MacLachlan, J.C., and Kleinkopf, 1969. Configuration of the crystalline basement rocks of Colorado RMAG Research Committee.

MacQuown, W.C., 1945. Structure of the White River Plateau near Glenwood Springs Colorado. GSA Bull. Vol. 56, No. 10.

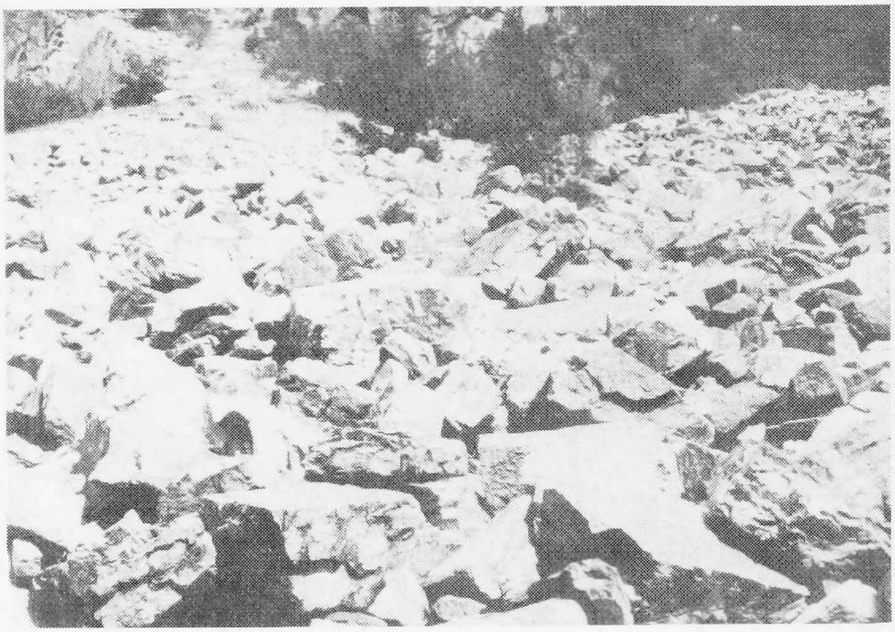


Fig. 10 - Blocky, coarse talus near French Creek.

Fig. 11 - Debris fan at Tie Gulch restricts the channel to approximately half its normal width.

GEOTEXTILE EARTH REINFORCED RETAINING WALL TESTS:

Glenwood Canyon, Colorado

by

J. R. Bell
Professor of Civil Engineering
Oregon State University
Corvallis, Oregon

R. K. Barrett
District Geologist
Colorado Division of Highways
Grand Junction, Colorado

A. C. Ruckman
Geotechnical Engineer
Corn Construction
Grand Junction, Colorado

for

Presentation

62nd Annual Meeting

Transportation Research Board

Washington, D.C.

January, 1983

The use of product, trade, proprietary, and company names in this paper is for clarity and does not imply endorsement or superiority to similar items.

Editor's note: Figures 2 and 7 referred to in this paper are not included due to the non-reproducible quality of the submitted copy.

GEOTEXTILE EARTH REINFORCED RETAINING WALL TESTS:

Glenwood Canyon, Colorado

INTRODUCTION

The Colorado Division of Highways designed and constructed a geotextile earth reinforced retaining wall in conjunction with Project I 70-2(90) in Glenwood Canyon, Colorado. This is one of four experimental flexible walls constructed on this project. The other wall systems are all proprietary, including Wire Wall®, Retained Earth® and Reinforced Earth®. Construction was completed during the spring of 1982.

Site Description

Glenwood Canyon is a narrow, steep-walled chasm cut by the Colorado River through resistant limestone, quartzite and granite. The deep slash through the bedrock was formed by a gradual regional uplift, causing the Colorado River to accelerate downcutting, with limited lateral cutting. The 12-mile long canyon is located about 150 miles west of Denver in west-central Colorado as shown in Figure 1.

Geologic investigations show that bedrock lies up to 150 feet below the river, and that thick lake deposits consisting of highly compressible silts and clays are present through the east half of the canyon. The lake deposits indicate that at one time a temporary dam was formed at some point in the canyon.

The Problem

The damming of the river was probably the result of a catastrophic event. Based on Carbon-14 dating, lacustrine deposition occurred over an approximate 2,000 year period, ending about 7,000 years ago. This segment of the canyon's history continues to be a unique and interesting puzzle for project geologists. It has certainly presented unique and difficult problems for foundation designers; as, it was assumed in early design phases that bedrock would be found at shallow depths beneath the river, and that foundations would not be a problem in the otherwise severely constrained interstate corridor.

The first project designed for the area where the compressible deposits were found included a rigid, post-tensioned cantilever wall with a cantilever pavement section (1)*. This design was obviously incompatible with the geology. Geologists and engineers could safely predict that significant differential settlements would occur, but predicting the amount of settlement and the time required were beyond the state of the art. Laboratory tests indicated a settlement range of 4 to 40 inches and settlement times of 6 months to 15 years.

Surcharging was not possible due to limited space and wick drains were deemed prohibitively expensive. Architects and designers finally relented on their insistance for rigid structures and allowed the use of flush-faced, flexible walls, with the stipulation that the first project be used as a full-scale test to determine field behavior of the lake deposits.

The only flexible retaining wall system fully approved for use by the Federal Highway Administration (FHWA) on the Colorado Interstate system was the Reinforced Earth Company product. With the possibility of extensive requirements for flexible walls in the canyon, it was decided that other systems should be tested to determine if one or more could be approved for

*Underlined numbers in parentheses refer to items in the Bibliography.

competitive bidding. The I-70-2(90) project was designated experimental to allow testing of these other systems and to monitor behavior of the lake deposits. Four wall types were tested: Wire Wall®, Retained Earth®, Reinforced Earth®, and geotextile walls.

This paper discusses design, construction, and performance of the geotextile wall tests.

Objectives of the Geotextile Wall Tests

Geotextiles have been model tested as earth reinforcement for a number of years (2)(3) and several walls have been successfully constructed (4)(5)(6). This system offers an apparently significant economic advantage over most proprietary systems. Still, there are enough questions remaining concerning design parameters and long-term stability to cause reluctance on the part of designers to fully accept this system.

None of the full-scale geotextile walls reviewed in the authors' research have shown significant creep and all have performed satisfactorily, even under loadings imposed by heavy logging trucks. However, none of the full scale tests were designed to determine limiting bounds for geotextiles as earth reinforcement. Many of these walls had high live loadings; therefore, dead loads did not produce low factors of safety, and creep resistance was not really tested.

A primary objective of the Glenwood Canyon test is to determine lower stability limits for a geotextile earth reinforcement systems. This is investigated by designing at or near equilibrium safety factors on portions of the walls to test the reliability of the current design procedures.

A second objective is to demonstrate that the system can be constructed by a major contractor. Most other walls had been the products of special crews, causing labor and other costs to be disproportionately high.

A third objective is to demonstrate over-all cost effectiveness of the geotextile reinforcement system when directly compared to other systems. With several systems on the same project and erected by the same contractor, a more reliable cost comparison is possible.

Fourth, to investigate tolerance to differential settlement.

The Fifth objective is to demonstrate a facing system that could perform for the design life expectancy of a wall system.

Finally, by reduced fabric embedment lengths in the lower portion of the wall, to demonstrate the stability of this system in side-hill and other situations where minimizing initial excavations could save a significant amount of money.

TEST DESCRIPTION

The geotextile test wall is approximately 15 feet high and 300 feet long. The wall is divided into ten 30 foot segments, with a different fabric or fabric strength combination used to construct segments one through eight. Nine and ten are identical to one and two respectively except the lower fabric layers are shortened.

The segments were designed with different factors of safety. Six segments had very low computed safety factors and were expected to creep, possibly to failure.

Geotextiles

Four common, readily available nonwoven geotextiles were selected for the tests. Each fabric was used in two weights. These fabrics represent a range of fabric constructions, polymers, and stress-strain characteristics. None of the geotextiles have particularly high strength or other special characteristics. The test geotextiles and some descriptive characteristics are listed in Table 1.

The tensile strengths of the geotextiles were determined by a wide-strip tensile test (7). Specimens 8 inches wide and 4 inches long between grips were loaded in simple tension at a constant rate of strain of 10%. The specimens were soaked in water prior to testing. The peak strengths and corresponding elongations as determined by these tests are tabulated in Table 2. Typical load-strain curves are shown as Figure 2.

The recommended working loads in Table 2 are selected to prevent failure by tertiary creep. The various phases of creep are illustrated in Figure 3. Tertiary creep can be prevented by limiting the applied loads to values less than some critical value below which tertiary creep will not occur during the design life. Experience and tests show this critical value is controlled, for a given geotextile, by loading and environmental conditons. The recommended values were suggested by tests performed at Oregon State University (8).

Backfill Soil

The backfill soil was a free draining pit-run, rounded, well graded, clean sandy gravel. Nearly all particles were less than 6 inches. Approximately 50% passed the 3/4" U.S. Standard Sieve and about 30% passed the #4 sieve. The backfill soil is illustrated in Figure 6. Compaction specifications required 95% of AASHTO T-180. Values of 35° and 130 lb/ft^3 were assumed for the angle of internal friction and unit weight respectively as conservative values for preliminary design calculations.

Instrumentation

Instrumentation at the site was designed to provide both qualitative and quantitative information on settlement in the vicinity of the wall and to identify specific soil layers or zones of settlement. Information on horizontal deflection in the foundation soils and the vertical deflections of the face of the wall was obtained. Measurements of the deflections of the wall face and the surface above the wall were made to indicate settlement and creep of the fabrics. Movements within the backfill soil mass were also monitored.

These measurements were taken with 5 vertical inclonometer/Sondex® installations spaced evenly in front of the wall 5 feet out from the face; 5 manometers installed evenly along the back edge of the wall; 30 horizontal inclinometer/extensometer casings, 3 per test segment, spaced vertically in the center of each segment; direct measurement survey posts at the center of each segment; and several survey points on and in front of the wall for direct measurement of changes in elevation. Many of these installations can be identified in Figure 7.

Test Wall Design

The test wall was designed by assuming the fabric layers had to resist a triangular lateral pressure distribution by friction on the portion of the fabric layer extending beyond the Rankine failure surface. This method was used by Lee, et al. (9) for Reinforced Earth® walls. The method was modified for fabric walls by researchers at Oregon State University (2)(10) and it has been used by the U.S. Forest Service (4)(5)(11) and the New York Department of Transportation (6), to construct several successful geotextile walls in the United States.

The geotextile wall section showing the fabric layer spacings is presented in Figure 4. The general wall layout is given in Figure 5 which shows the ten wall segments and identifies the fabric types in each. This figure also shows the locations of some of the instrumentation installations.

Each test segment is 30 feet long. The fabric layers all extend into the fill 12 feet except for the lower layers in Segments 9 and 10 (Figure 4). All segments incorporate a stronger geotextile (Trevira®1155) for the lower 3 layers. Segments 5, 6 and 8 incorporate a light weight fabric for the upper layers (10 through 17) and a heavier weight of the same fabric for the lower layers (4 through 9). All other segments used one strength geotextile except for the cover fabric (Layer 18) and the lowest layers (1 through 3).

To facilitate construction, geotextile layer spacings are the same in all wall segments. Therefore, since the geotextiles have different strength characteristics the factors of safety are different for different segments. Design relative loads for each wall segment are shown in Table 3. The "relative load" is the computed load expressed as a percentage of the appropriate ultimate load (strength) listed in Table 2.

Safety considerations dictated that the wall not fail rapidly during construction; therefore, some conservatism was retained in the design method. Table 3 shows, however, that many computed loads are well above the recommended values in Table 2 and in two segments the loads approach the ultimate loads. It was expected from this design that the wall would exhibit significant strains in some fabric layers and deflections of the wall would be evident. In some segments, failure by tertiary creep was considered a real possibility.

CONSTRUCTION

Construction procedures used in the Glenwood Canyon wall were modeled after U.S. Forest Service guidelines (5). Figure 6 illustrates in detail the steps required to complete each lift. Figure 7 is a collection of photographs of the actual construction. Figure 7(a) shows the partially completed wall with the forms set for the next lift (Step 1, Figure 6) and the fabric spread (Step 2, Figure 6). In Photo 7(b) the backfill for the new layer is being placed. In Figure 7(c) the backfill is being spread (Step 3, Figure 6) and the windrow built (Step 4, Figure 6). Figure 7(d) shows the fabric folded back over the windrow (Step 5, Figure 6) and the backfill lift being completed (Step 6, Figure 6).

Based on field observations during construction, few modifications for construction of future walls would be made to the general plan used for the test series. It would be prudent to design thin (6"-9") lifts for the first 2 to 3 feet, to allow the field crew to become familiar with the technique. It takes a new crew 3 or 4 lifts to develop their technique so they can obtain a uniform face.

The face forming system used on this project worked very well. However, the experience suggests it is only reasonable for lifts up to about 15 inches. Thicker lifts would require a different or modified forming system.

Maintaining design batter, or face slope, requires continual monitoring. The test wall series was planned for a 1 in 10 batter and was constructed with a 1 in 5 batter. This was due to a variety of factors, chief of which was failure to formally survey each lift.

Many geotextiles must be protected from the sun. Ultra-violet light is the only agent in a typical construction environment that will cause deterioration of either polypropylene or polyester. On this project, new lift faces were sprayed within 5 days with a low viscosity water-cement mixture. This fluid penetrated the fabric and set to a brittle stiffness. It bonded well and provided excellent protection, even for the smoother fabrics.

Facing

The wall was faced with gunnite. This facing was easily applied by an experienced crew, and has withstood differential settlements of about 12 inches over 300 feet in only 3 months with very little cracking of the surface. About 65 cubic yards of gunnite were required for the approximately 4700 square feet of wall face.

A number of facades could be adapted for geotextile walls including logs, treated timber, vertical or horizontal boards, or precast concrete panels. Any of these systems could be developed cost competitively on a major wall. The wall would be constructed as described previously and the facing attached to the completed wall as a free standing facade tied back to the geotextile wall.

Cost

Cost for the series of geotextile test walls ranged between $11.00 and $12.50 per square foot of wall face. Variability in the cost figure depends on which items were excluded as special research features and which were included as required for all walls. Cost breakdown in terms of completed square foot of wall face for 1982 prices was as follows:

Geotextiles	$2.00-$2.50
Labor	$ .50-$1.00
Equipment	$ .50-$1.00
Backfill	$5.00 (includes haul)
Gunnite	$3.00
	$11.00-$12.50

DISCUSSION

In general the construction procedures, costs, and wall performances have been very satisfactory. In fact, performance to date has been better than expected in that large strains anticipated in some segments of the walls have not occurred. There are several possible reasons for this fact: (1) the instrumentation does not accurately indicate the strains in the geotextiles (2) the assumed backfill parameters are incorrect, (3) the theory does not accurately model the true mechanisms, and (4) the laboratory geotextile tests used do not adequately indicate the in-soil behaviour of the geotextiles. Discussions of each of these follows.

Model tests (2) have shown that the failure zones in reinforced walls are very narrow. Therefore, the zone of significant strains (initial and creep) in the geotextiles may be narrow. In the test wall the fabric strains may be masked by settlement, at least until strains become very large. To date (November, 1982) the instruments have not shown fabric strains to be occurring.

Values of 35° and 130 lb/ft^3 were assumed for backfill friction angle, ϕ, and unit weight, γ, respectively. These values were used to calculate the geotextile relative loads shown in Table 3. The maximum design load for each wall segment is tabulated in Table 4. These values show that according to the design assumptions for the critical layers in Segments 4 and 8 the factors of safety against an immediate failure were 1.0 (a relative load of 100 equals a factor of safety of 1.0). Other segments had immediate factors of safety of 1.25 or higher. With respect to creep failure (rupture by tertiary creep) all but Segments 2, 3, and 10 had factors of safety much less than 1.0. Segment 5, for example, had only a factor of safety of 0.5 (ratio of relative load to allowable road). Obviously, the design calculations were conservative as failure has not occured.

As-built, Ø was about 42° and the average unit weight about 135 lb/ft³. When these values are used with the other design assumptions, maximum relative loads are as shown in Column 3 of Table 4. When these values are considered, the factors of safety against an immediate failure are all greater than 1.0 but factors of safety against creep are still considerably less than 1.0 for six of the ten wall segments. There are apparently more conservative factors than just the assumed backfill parameters.

A major question is the validity of the theory used. The theory does not actually analyze a composite reinforced material. It is really a pseudo tie-back analysis. It is known to be conservative (2) but how conservative is not known. The analysis in this design assumed the at-rest earth pressure Ko rather than the active pressure Ka. The at-rest pressure has also been used for similar analyses by others (9)(10)(5). The use of the at-rest pressure was certainly reasonable when the heavy traffic loads were expected but may be excessively conservative for the actual case with dead loads only. The high elongation geotextiles may encourage the development of the active state. When the maximum geotextile relative loads are further reduced for active lateral earth pressure, the values in Column 4 of Table 4 are obtained. Now, none of the computed loads exceed the allowables and creep failures are not indicated; however, creep greater than that indicated by the field instrumentation would still be expected.

The distribution of the total load among the geotextile layers is another question. The calculations assume the soil stresses increase linearly with depth and the load on a given fabric layer is equal to the sum of the soil stresses over an area bounded by the mid-distances to the layers above and below. Field measurements have shown the actual loads to be more uniform with depth (12)(13). Further, for high elongation geotextiles such as those used in this study the more highly stressed fabric layers may yield suffcently to transfer parts of their loads to adjacent layers. It might be more realistic to average the geotextile layer loads. Averaging assumes no layer can fail unless all layers fail. This assumes that if the load on any layer exceeds the load on adjacent layers that layer will creep more than adjacent layers, thereby, transferring loads to the adjacent layers.

Average geotextile relative loads are shown in Table 5. For these values, there is no concern about possible immediate failure, however, for the at-rest case when the lower portion of the wall is considered creep failure is still critical for six of the 10 segments. Only when the loads for the active pressure case are averaged are the results consistent with field strain measurements on the test wall. At these stress levels the fabrics would strain approximately 10% or less. This is the maximum strain in a layer and not an average. The instrumentation may not detect this magnitude of movement as being due to fabric creep rather than settlement.

These discussions suggest that the theory greatly over estimates stresses in the geotextiles and that past practices have been excessively conservative. Before, accepting this conclusion; however, it is necessary to consider the problem of establishing allowable geotextile loads from simple, in-isolation, tensile testing.

It is known that when confined in a soil the load-strain-time relationships of geotextiles are changed (14)(15). For the needle punched fabrics these changes are great. The initial strains at relative loads of 20-40% may be reduced by one half or more (14). The ultimate strengths, however, are only slightly increased. There is limited evidence, also, that the critical stress to initiate tertiary creep failure is not greatly increased (14)(16). The creep rate at intermediate relative loads is, however, decreased by orders of magnitude. Therefore, the critical loads assumed in this study may be reasonable but confinement of the geotextiles in the soil may have reduced initial strain and delayed the development of time dependent strain. It may be that the higher computed loads, at least as high as the averages, are realistic and that only time is required to develop the expected high strains.

The wall instrumentation monitoring will be continued to observe future trends. Also, wall Segments 5 and 6 have been surcharge loaded. This may help to clarify the situation; however, there is real need for full scale, instrumented walls, designed to fail. Only by going all the way to failure with different types of geotextiles for different loadings will it be possible to verify or modify design methods and select appropriate safety factors.

The surcharge load was added too recently to provide data for this paper. These results will be analyzed in a future report.

Using stronger fabrics will allow building walls with fewer, thicker lifts. This appears to offer an economic benefit because of reduced labor. However, experience in these tests suggests that the forming method used is not generally practical for lifts greater than about 15 inches. Thicker lifts will require some different method. A more sophisticated method will probably not be economical except on very large jobs. The use of strong geotextiles may not, therefore, offer significant savings except in special cases or for high walls where they would be used in the lower layers.

Some fabrics are stronger in the machine direction than in the cross-machine direction. However, to utilize this extra strength it is necessary to cut the fabric roll into many short sections and sew them together before installation. This is probably not generally cost effective. It is probably best to use the cross-machine strength in design. This must be considered in sampling and testing geotextiles.

While geotextile costs are not to be ignored, the cost data for this wall show geotextile costs are a small percentage of total wall costs. If excavation is required, this percentage cost becomes even smaller. Therefore, using conservative allowable loads and other conservative design assumptions does not unduly increase costs.

CONCLUSIONS

The results of this study suggest the following conclusions.

1. Geotextile reinforced walls are economical and practical.
2. Construction procedures are practical for large contractor constructed projects.
3. Suitable simple, economical, durable facings can be adapted for this system.
4. The mechanisms of geotextile reinforced soil are not well defined. Further, the stress-strain-creep behaviors of geotextiles embedded in soils are not fully understood and present (1983) ability to evaluate geotextiles, and rationally select allowable loads is limited.
5. Designs based on the Rankine model using at-rest earth pressures as

previously published will give economical, safe, practical designs for the following conditions.
   a. Cohesionless backfill
   b. Maximum allowable relative loads as specified in this report to limit creep.
   c. Allowable loads may be applied to average loads computed as indicated in this paper for high elongation geotextiles.
   d. Factor of safety of 1.5.
6. Conservative interpretation of geotextile strengths has limited effect on economy as cost of the reinforcing component is a small part of the total wall cost.
7. Full scale wall tests to failure are needed to clarify the theory; develop appropriate factors of safety; and, therefore, allow more rational, and perhaps, more economical designs.
8. Reduced fabric lengths in the lower portions of fabric walk may be a safe, effective means of minimizing excavations in side-hill installations.

ACKNOWLEDGEMENTS

The test walls were constructed during the spring of 1982 with Federal Highway Administration (FHWA) Task Order funds, Federal Aid Interstate (FAI) funds and State of Colorado funds. Federal Aid was denied for wall test segments that were designed at lower bound safety factors.

The wall was designed, instrumented and monitored as a joint venture by Colorado Department of Highways Research and Development, Central Lab, and District Materials personnel.

Dupont Co., Hoechst Fibers, Crown Zellerbach Co, and Phillips Fibers Corp. provided the geotextiles for the test wall.

The wall was constructed by Peter Kiewit and Sons Construction Company.

BIBLIOGRAPHY

1. Trapani, R.J. and Beal, E.A., "Glenwood Canyon I-70 Environmental Concern," Preprint 82-025, ASCE, Las Vegas, April 1982.

2. Bell, J R., Stilley, A.N. and Vandre, B., "Fabric Retained Earth Walls," Proceedings, 13th Annual Engineering Geology and Soils Engineering Symposium, Moscow, ID., 1975.

3. Holtz, R.D., "Laboratory Studies of Reinforced Earth Using a Woven Polyester Fabric," Proceedings, International Conference on the Use of Fabrics in Geotechnics, Ecole Nationale Des Ponts et Chaussees, Paris, Vol. III, April 1977, pp. 149-154.

4. Bell, J R. and Steward, J.E., "Construction and Observations of Fabric Retained Soil Walls," Proceedings, International Conference on the Use of Fabrics in Geotechnics, Ecole Nationale Des Ponts et Chaussees, Paris, Vol. 1, April 1977, pp. 123-138.

5. Steward, J.E., Williamson, R., Mohney, J., "Guidelines for use of Fabrics in Construction and Maintenance of Low Volume Roads," FHWA TW-78-205, U.S. Federal Highway Administration, Washington, D.C., 1977.

6. __________, Case Histories, Reinforced Earth Walls, Earth-Fabric Wall, New York State Department of Transportations, Soil Mechanics Bureau, Albany, NY, March 1981.

7. Shrestha, S.C. and Bell, J R., "A Wide Strip Tensile Test for Geotextiles, Proceedings, Second International Conference on Geotextiles, Las Vegas, Vol. III, August, 1982. pp. 739-744.

8. Allen, T., Vinson, T.S., and Bell, J R., "Tensile Strength and Creep Behavior of Geotextiles in Cold Regions Applications," Proceedings, Second International Conference on Geotextiles, Las Vegas, Vol. III, August 1982, pp. 775-780.

9. Lee, K.L., Adams, B. D. and Vagneron, J. M. J., "Reinforced Earth Retaining Walls," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 99, No. SM10, 1973 pp. 745-763.

10. Whitcomb, W. and Bell, J R. "Analysis Techniques for Low Reinforced Soil Retaining Walls and Comparison of Strip and Sheet Reinforcements," Proceedings, 17th Engineering Geology and Soils Engineering Symposium, Moscow, ID, 1979.

11. Steward, J. and Mahoney, J., "Trial Use Results and Experience Using Geotextiles for Low-Volume Forest Roads, Proceedings, Second International Conference on Geotextiles, Las Vegas, Vol. III, August, 1982, pp. 569-574.

12. John, N., Johnson, P., Ritson, R. and Petley, D., "Behaviour of Fabric Reinforced Soil Walls, Proceedings, Second International Conference on Geotextiles, Las Vegas, Vol. III, August, 1982, pp. 569-574.

13. Hausmann, M.R., "Behavior and Analysis of Reinforced Soil," Ph.D. Thesis, University of New South Wales, Kensington, Australia, 1978.

14. McGown, A, Andrawes, K.Z., and Kabir, M.H., "Load-Extension Testing of Geotextiles Confined In-Soil," Proceedings, Second International Conference on Geotextiles, Las Vegas, Vol. III, August, 1982. pp. 793-798.

15. Andrawes, K.Z., McGown, A., Mashhour, M.M. and Wilson-Fahmy, R.F., "Tension Resistant Inclusions in Soils," Journal, Geotechnical Engineering Division, ASCE, Vol. 106, No. GT12, December 1980.

16. Allen, T., "Application of Geotextiles in Cold Regions," M.S. Thesis, Oregon State University, Corvallis, OR 1983.

TABLE 1 - TEST GEOTEXTILES

"Trade Name" (Manufacturer) CDOH Code No.	Approximate Weight (oz/yd²)	Filament	Construction
"Fibretex"® (Crown Zellerbach)		Continuous, Polypropylene	Needle punched
CZ 200	6		
CZ 400	12		
"Supac"® (Phillips Fibers)		Staple, Polypropylene	Needle punched, heat bonded (one side)
P 4 oz	4		
P 6 oz	6		
"Trevira"® (Hoechst Fibers)		Continuous, Polyester	Needle punched
H 1115	5		
H 1127	11		
"Typar"® (DuPont)		Continuous, Polypropylene	Heat bonded
D 3401	4		
D 3601	6		

TABLE 2 - GEOTEXTILE STRENGTH CHARACTERISTICS

Geotextile	Ultimate Strength lb/ft	Failure Strain %	Recommended Working load lb/ft (% of Ultimate)
CZ 200	400	140	220
CZ 400	680	145	375
			(55)
P 4 oz	860	65	345
P 6 oz	1665	60	670
			(40)
H 1115	455	80	295
H 1127	1155	75	750
			(65)
D 3401	525	60	210
D 3601	850	55	340
			(40)

TABLE 3 - DESIGN RELATIVE LOAD FOR TEST WALL SEGMENTS

Wall Segment	Relative Load* (%) 1	2	3	4	5	6	7	8	9	10
Layer										
17	10	7	5	13	16	19	10	21	10	7
16	16	12	8	20	26	30	16	34	16	12
15	17	13	9	21	28	32	17	36	17	13
14	21	15	11	26	34	39	21	44	21	15
13	25	18	12	31	40	46	24	52	25	18
12	28	21	14	35	46	53	28	60	28	21
11	32	23	16	40	52	60	32	68	32	23
10	42	31	21	52	68	78	41	89	42	31
9	54	40	28	68	54	40	53	68	54	40
8	56	41	28	70	56	41	55	70	56	41
7	53	39	27	66	53	39	52	66	53	39
6	53	39	27	67	53	39	53	67	53	39
5	57	42	29	71	57	42	56	71	57	42
4	79	58	40	99	79	58	78	99	79	58

*Relative load is the computed fabric load expressed as a percentage of the geotextile ultimate strength.

TABLE 4 - COMPUTED RELATIVE LOADS AND RECOMMENDED ALLOWABLE GEOTEXTILE LOADS

Wall Segment	Computed Maximum Relative Load (%)			Allowable Load %
	Design	As-Built At-Rest (Ko)	As-Built Active (Ka)	
1	79	63	39	40
2	58	46	29	65
3	40	32	20	40
4	99	79	49	55
5	79	63	39	40
6	78	63	39	65
7	78	62	39	40
8	99	79	49	55
9	79	63	39	40
10	58	46	29	65

TABLE 5 COMPUTED AVERAGE RELATIVE LOADS AND RECOMMENDED ALLOWABLE LOADS

Wall Segments	Computed Average Relative Load* (%)			Allowable Load(%)
		As-Built		
	Design	At Rest (Ko)	Active (Ka)	
1	39(59)	31(47)	19(29)	40
2	29(43)	23(34)	14(21)	65
3	20(30)	16(24)	10(15)	40
4	49(74)	39(59)	24(37)	55
5	47(59)	38(47)	23(29)	40
6	44(43)	35(34)	22(21)	65
7	38(59)	30(47)	19(29)	40
8	60(74)	48(59)	30(37)	55
9	39(59)	31(47)	19(29)	40
10	29(43)	23(34)	14(21)	65

*Averages of layers 5 through 17. Numbers in parentheses are averages of layers 4 through 9.

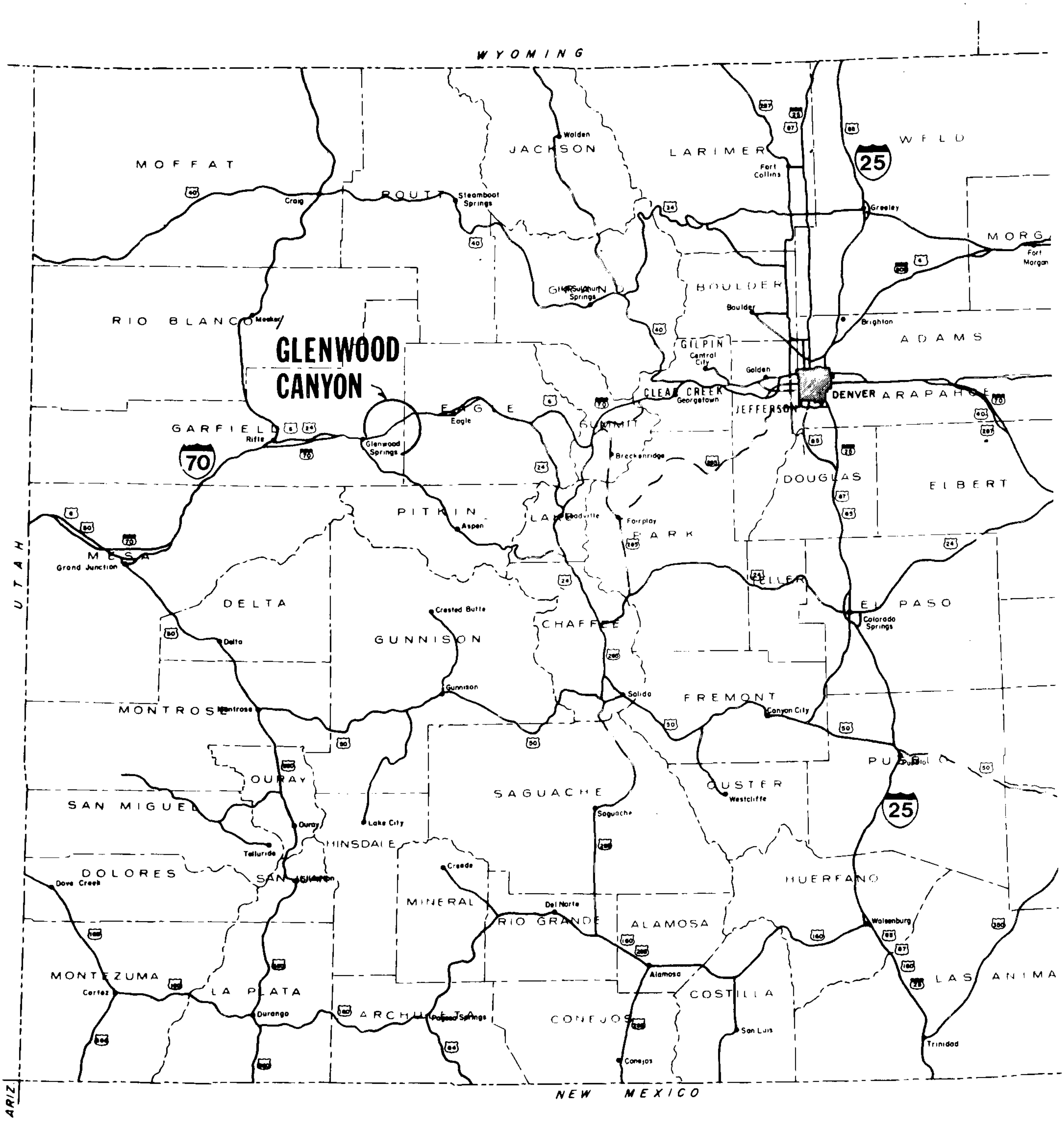


FIGURE 1   Map of Western Colorado showing Glenwood Canyon Location.

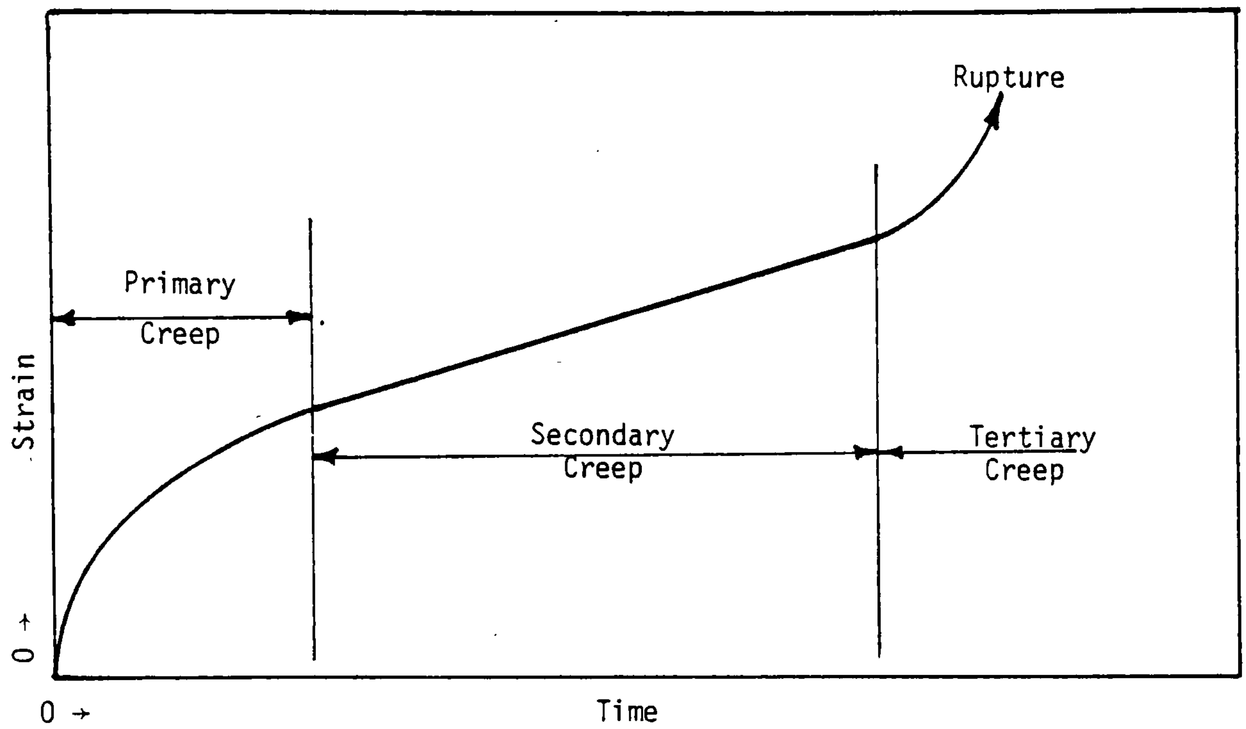


FIGURE 3 Phases of Creep for a Typical Geotextile Tested in Isolation at Constant Load and Temperature.

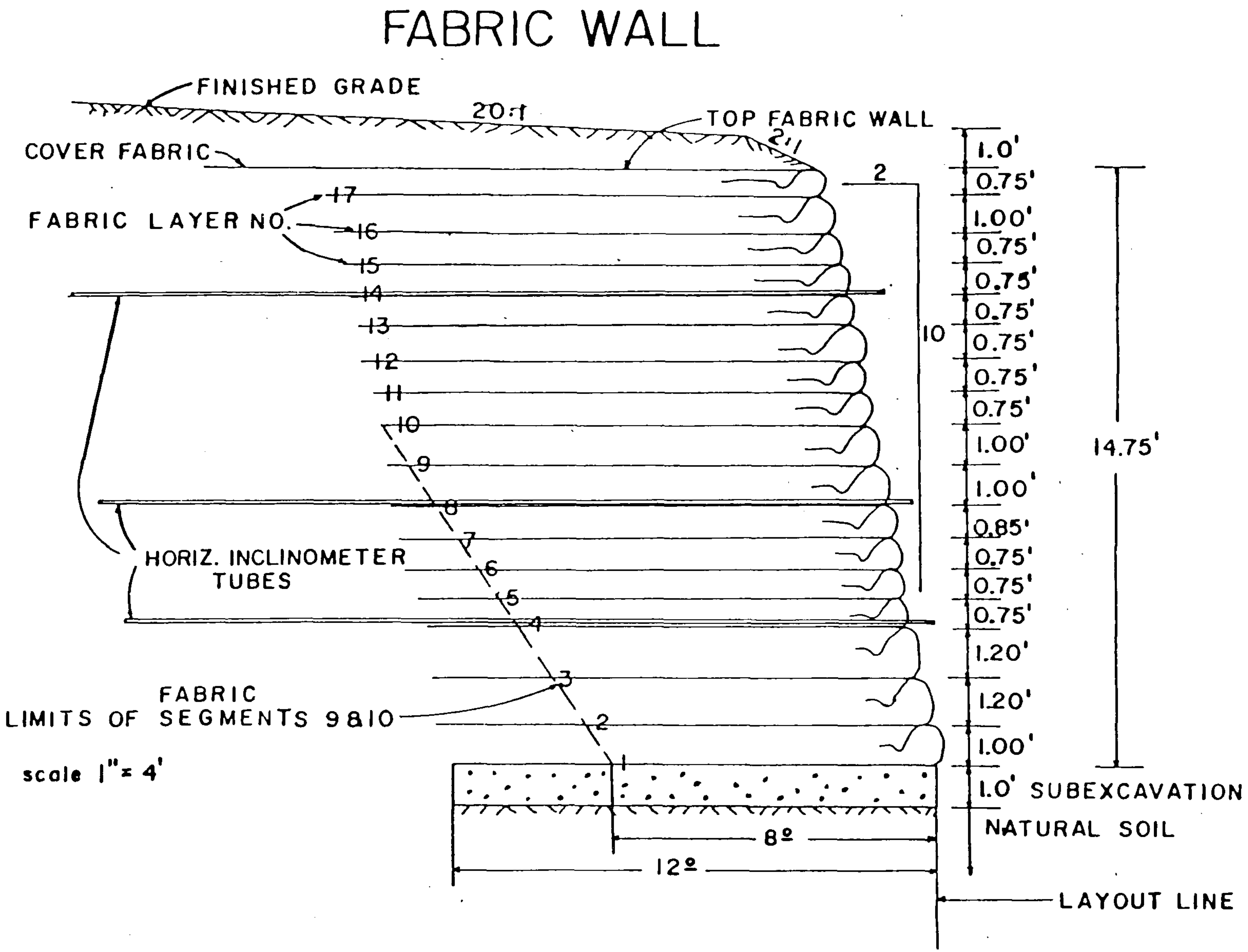


FIGURE 3 - GEOTEXTILE WALL SECTION

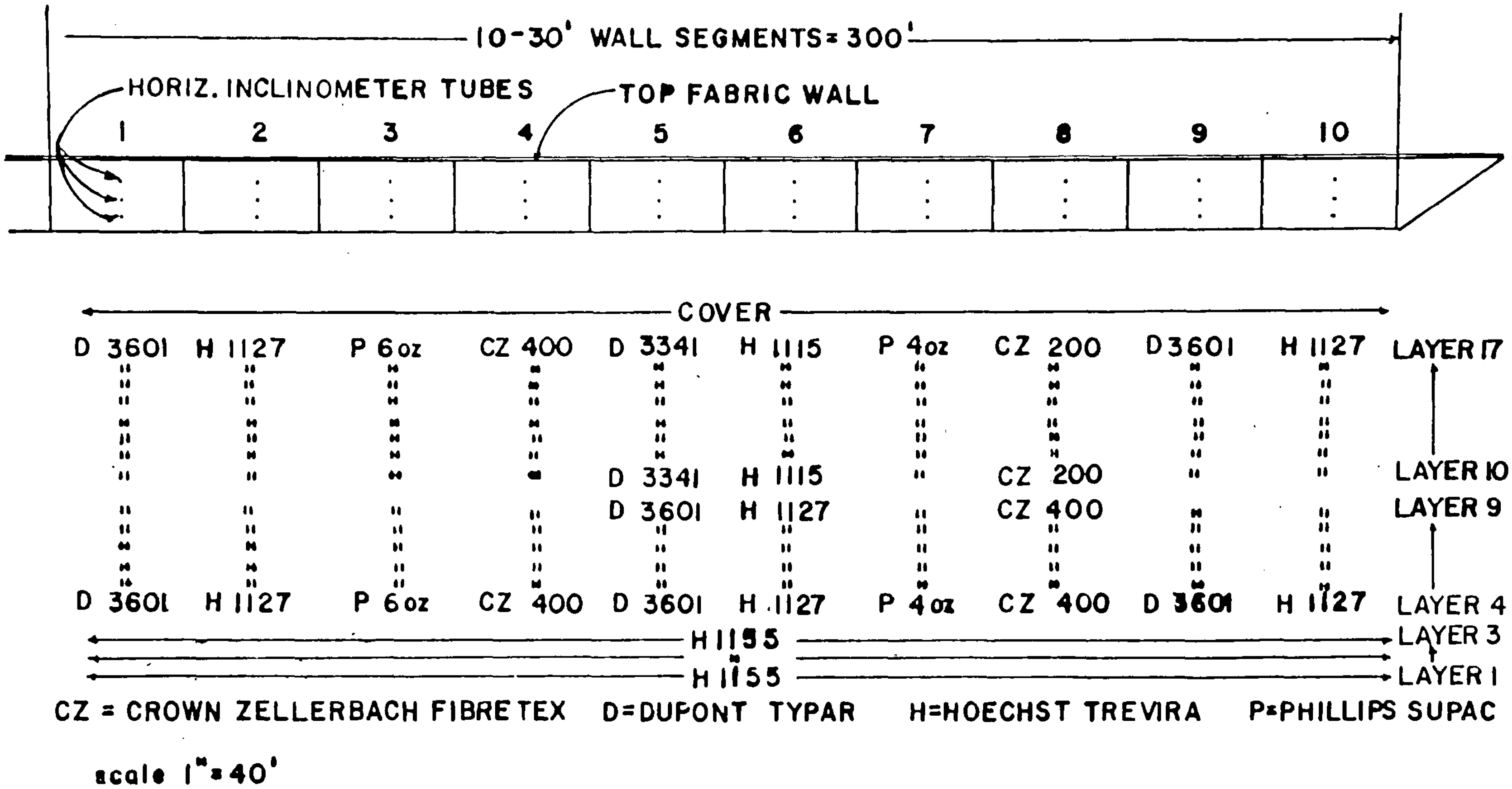


FIGURE 4 - GEOTEXTILE WALL ELEVATION SHOWING TEST SEGMENTS AND FABRIC TYPES

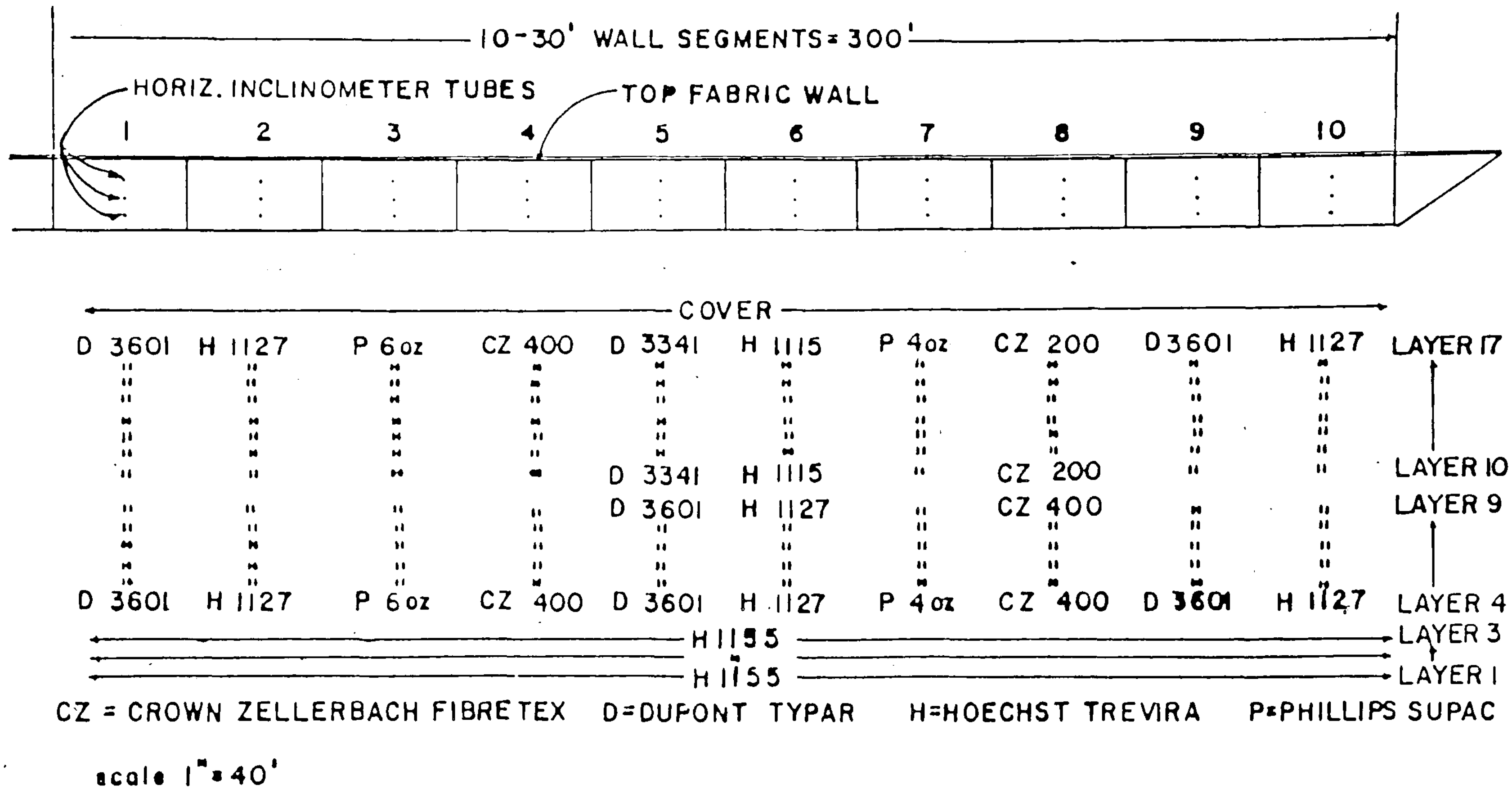
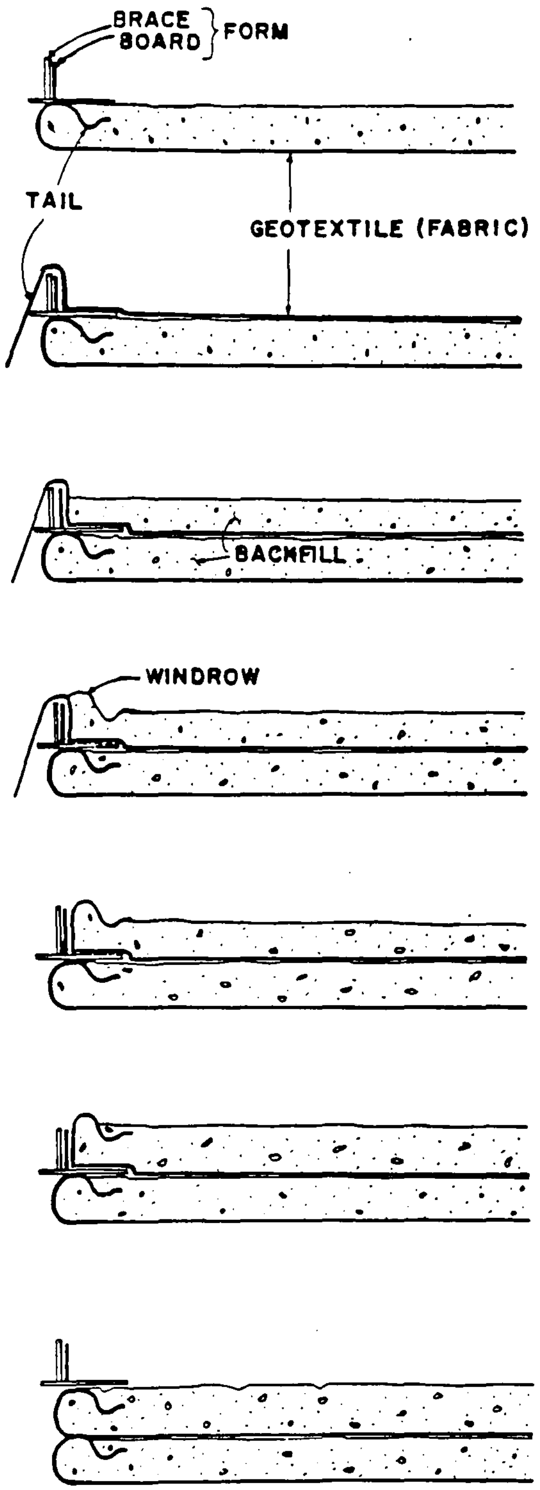


FIGURE 5 GEOTEXTILE WALL ELEVATION SHOWING TEST SEGMENTS AND FABRIC TYPES



1. SET FORM ON COMPLETED LIFT
2. UNROLL FABRIC AND POSITION SO THAT A 3± FOOT WIDE "TAIL" DRAPES OVER THE FORM
3. PLACE BACKFILL TO ABOUT HALF OF THE TOTAL LIFT HEIGHT
4. PLACE A WINDROW TO SLIGHTLY GREATER THAN FULL LIFT HEIGHT AGAINST THE FORM
5. PLACE THE FABRIC "TAIL" OVER THE WINDROW AND LOCK INTO PLACE WITH BACKFILL
6. COMPLETE BACKFILLING FOR PLANNED LIFT THICKNESS
7. RESET THE FORM AND REPEAT THE SEQUENCE

FIGURE 6 General Geotextile Wall Construction Procedure.

Geothermal Heating of the Bridges and Tunnels in Glenwood Canyon

by

Kynric Pell

and

John Nydahl

University of Wyoming

and

Denis Donnelly, Herbert Swanson and Richard Griffin

Colorado Department of Highways

Introduction

During the past three years the Colorado Department of Highways has investigated the feasibility of incorporating geothermal heating in the pavement at critical locations of Interstate 70 currently under construction through Glenwood Canyon. Roadway ice has been a problem on the existing two lane road since some areas of the roadway receive no direct sunlight during the winter and the Colorado river flowing next to the road causes the humidity to be very high. This problem is expected to be exacerbated on the new highway which will include several tunnels and considerable elevated structure.

A review of an exploration of the existing geology and geothermal activity in Glenwood Canyon is available in reference 1.

Several thousand gallons per minute of geothermal water are available from existing springs at either the east or west end of the canyon; however, this water would have to be transported as much as five miles to heat the various structures. In spite of the distance involved, thermal loss from the water and pumping costs are both within reason. It is clear that the geothermal water could not be circulated within the pavement since a system failure could lead to freezing of the water and destruction of the pavement. In addition the geothermal water contains both corrosive and fouling elements which could require maintenance on the exposed plumbing elements. It was therefore decided to test ammonia charged, gravity operated heat pipes as intermediate heat exchangers to prevent these problems from occurring within the deck slab itself. Ammonia heat pipes were chosen because of their simplicity and success in similar applications (2-5). The heat pipes were tested in an experimental facility designed especially for this purpose.

Experimental Facility

The heat pipes tested in the facility are illustrated schematically in Figures 1 and 2. Any time the flowing water is warmer than the deck, energy from the geothermal water is conducted through the evaporator pipe, vaporizing a portion of the liquid ammonia in the evaporator. The ammonia vapor rises into the condenser tubes where it condenses, giving up the heat of vaporization to the deck and condensate then returns to the evaporator under the influence of gravity. Since the energy transport is in the form of the latent heat of vaporization, the temperature drop across the heat pipe is extremely small.

The two different heat pipe designs depicted in Figures 1 and 2 were provided by the SETA Corporation of Laramie, Wyoming and Energy Engineering, Inc. of Albuquerque, New Mexico. The SETA Corporation provided three manifolds with condenser element spacings of 0.15 m (6"), 0.3 m (12") and 0.46 m (18"). These three units will be referred to as the SETA 6", SETA 12" and SETA 18" modules respectively. EEI provided heat pipe manifolds of an alternate design with 0.15 m (6"), EEI 6", and .20 m (8"), EEI 8", condenser pipe spacings.

A prototype bridge which was composed of two Twin-T beams placed on two meter high abutments was erected in Glenwood Springs Engineering and Maintenance Yard of the Colorado Department of Highways. Longitudinal and transverse layers of reinforcing steel which supported the heat pipe manifolds were placed upon the 15 m x 5 m (50' x 16') deck formed by the Twin T's. This deck was then overlaid with 0.15 m (6") of concrete with the top of all the condenser elements located approximately 6.4 cm (2.5") below the surface of the deck. Heat pipes covered the entire deck with the exception of an unheated 2.5 m x 5 m (8' x 16') control section. The bottom of the deck was insulated with 5 cm (2") of foam except under the control section. A cross section of the experimental facility is depicted in Figure 3.

The geothermal water was obtained from a dammed drainage ditch that ran through the maintenance yard. Some of the water had to be recirculated through the heating system at the higher flow rates since the spring's output was not adequate.

In order to quantify the performance of the heating system, a variety of transducers were employed. The flow rate of the geothermal water was monitored in addition to the temperature of the incoming and exiting water for each manifold section. The temperatures on some of the condenser pipes, the upper and lower surfaces of the deck, as well as some points midway between the condenser elements were monitored with the arrangement shown in Figure 4 for each of the five manifold sections. In addition, the local environment was quantified in terms of: ambient air temperature, wind speed, wind direction, relative humidity, barometric pressure, and incoming solar radiation. All temperatures and environmental data were recorded at one minute intervals using a 100 channel data acquisition system supplied by the Mechanical Engineering Department of the University of Wyoming. The surface condition of the deck was also photographed on a 24 hour basis using a time lapse camera activated at 10 minute intervals. A complete description of the test facility and results is available in reference 2.

System Performance

The geothermal heating system was activated on March 18, 1980, shut down on April 11 for the summer, reactivated on September 19, 1980 and permanently disassembled in June of 1981. The monthly averaged climatic record for the 1980-81 winter is compared in Table 1 to the long term climatological averages. Table 1 also tabulates the percentage of each month that experimental data were obtained. This table indicates that the air temperature was relatively mild in that the traditionally two coldest months of December and January were approximately 4°C warmer than normal. The coldest temperature recorded at the site was only -17.5°C. Twenty-two centimeters of snow were reported to have fallen by the Glenwood newspaper between Nov. 17, 1980 and April 30, 1981 which appears to be about average for that time period.

The monthly averaged geothermal water temperatures and flow rates for April 1980 and the 1980-81 winter are tabulated in Table 2 along with the corresponding average temperature increases that were induced on the five heated surfaces. Over the three month period December through February when the monthly averaged air temperatures were all near freezing, the following hierarchy in the averaged surface temperature increases was observed: EEI 6" (9.3°C) SETA 6" (6.5°C) SETA 12" (6.5°C) SETA 18" (5.8°C) EEI 8" (5.6°C). The monthly averaged flow rates and temperatures of the geothermal water during this period varied between 30 and 50 GPM and 24 to 25°C respectively. The above hierarchy indicates that the relative performance of the various designs did not completely follow what would have been anticipated but severe fouling in the water pipe had a large influence on the performance of these systems. This point will be pursued later.

Table 3 delineates the performance of the heat pipe systems in terms of various surface freezing characteristics over the life of the experiment. This table indicates that the heated surfaces were frozen between 1 and 15% as long as the unheated control which was at or below freezing 22% of the recorded time. A measure of the severity of the freezes is given by the area (degree-days) enclosed between the freeze line and surface temperatures below freezing on a temperature versus time plot. This freezing factor was reduced between 90% and 99% by the various heat pipe modules while the corresponding reduction in number of freeze-thaw cycles ranged from 66% to 95%. The photographic record shows the unheated control section was covered with snow or ice for approximately 545 hours and partly covered with snow for an additional 115 hours during the 1980-81 winter. The heat pipe modules reduced the duration of the above icy surface conditions by at least 96%. Figure 5 depicts one of these events where the control section was iced over as was the adjacent section of Interstate 70 while the heated surfaces on either side of the control section were all melted. The SETA 12" module can be seen on the left hand side of the frozen control section in this photograph while the EEI 6" module is on the right hand side.

In order to be able to predict the performance of these heat exchangers with various water temperatures, a coefficient of performance θ was defined as the ratio between the observed temperature increase of the heated surface (T_S - T_C) and the maximum possible temperature increase - the temperature difference between the geothermal water (T_W) and the unheated control (T_C). This coefficient of performance varies with the combined convective and radiative heat transfer coefficient between the heated surface and the environment, and with the system's overall specific heat transfer conductance u (w/m^2 °C) between the water and the heated surface. The variation in a system's conductance (u) during this experiment was mainly due to the effect of fouling and it was only slightly affected by the varying water flow rates.

The weekly average coefficient of performances for the SETA experimental modules are plotted versus time in Figure 6. This figure indicates that the θ for the SETA 6" module decreased from a value around 0.4 at the beginning of the experiment to a value around 0.23 one year later and that it had a 0.29 average over the coldest mid-winter months. The effect that fouling had on this pipe is indicated by the 20% drop in its θ from March through April after being cleaned on March 19, 1982 while the θ values of the other manifolds remained essentially constant or increased during this same period.

θ for the SETA 12" manifold varied between 0.21 and 0.43 and it had an average θ of 0.23 over the mid-winter months. The variation for the SETA 18" module was 0.13 to 0.29 with a 0.25 mid-winter mean. A detailed heat transfer analysis of these data and the corresponding environmental data indicated that the fouling had reduced the experimentally observed conductance of the SETA modules by at least 55%. The major effect of fouling on these units appears to have occurred fairly early in the experiment in that they maintained a reasonably constant conductance over the latter part of the experiment. Figure 6 indicates that the relative effects of the fouling on the SETA modules were such that there was very little difference in their performances after February 1981.

Figure 7 plots the coefficients of performance for the EEI modules and the SETA 6" module. θ for the EEI 6" manifold varied between 0.27 and 0.44 with a mid-winter mean of 0.39 while the EEI 8" unit's θ varied between 0.15 and 0.28 with a mid-winter mean of 0.24. This figure indicates that the EEI units maintained their relative performance over the experiment as opposed to the SETA modules which exhibited essentially no difference in their performances during the latter part of the experiment. The heat transfer analysis indicated that the conductances of the EEI modules had decreased by 60% over the whole experiment and was still decreasing at the end of the experiment due to fouling as opposed to the SETA modules which appear to have reached a steady state value.

Implementation

Even though the fouling of the experimental heat exchangers proved to be fairly extreme, all of the modules reduced their respective snow cover time by at least 96% relative to the unheated control.

A surface area of $59 m^2$ (640 ft^2) was heated by each of the heat pipe modules but this resulted in a very small water temperature drop even at flow rates as low as 35 GPM. A much larger series of heating units could have been installed with very little degradation in the performance of the last units. For example, if the control surface is frozen but dry and the water flow rate and temperature are 100 GPM and 38°C (100°F) respectively, the surface temperature of the first unfouled modules should be around 12°C (54°F) and the surface 1600 feet down the road should be around 8°C (46°F). For the same case but with fouling, the surface temperature above the first manifold should be 7.4°C (45°F) while the surface 1600 feet down the road would be at 6°C (42°F). It therefore appears that a large length of highway could be easily handled with the hot springs located in the canyon.

Corrosion of the water header pipe was very severe and the corrosion rates observed during these tests would limit the useful life of the heat pipe system utilizing similar water to approximately ten years. In addition deposition of minerals from the geothermal water created a sludge layer which reduced the thermal effectiveness of the heat pipes. The corrosion problem must be solved in order for a heat pipe system to be viable in this application. Maintenance could be employed to handle the deposition problem. The water did not attack the PVC pipes nor did fouling occur in the plastic pipes to any significant extent. This led to the initial hypothesis that a coating such as fusion bonded epoxy might be desirable inside the water header. Unfortunately most candidate materials have low thermal conductivities. Current efforts are centered on identifying suitable metallic alloys. The University of Wyoming and the Colorado Highway Department have entered into a joint project to identify appropriate materials for fabrication of the water header.

In the past year two pavement heating systems employing heat pipes of this design have been bid by SETA Corporation. Both of these bids have involved rather small surface areas on the order of 1000 m and have been between \$75.00 and \$83.00 per square meter of heated surface. It appears that large areas could be heated at a capital cost of \$65.00 per square meter exclusive of the piping and pumps required to deliver the water to the site to be heated.

Conclusions

Heat pipes have been demonstrated to be an effective means of snow and ice control on pavements. Systems of either design tested provide adequate surface fluxes for the Glenwood Canyon environment. The unrestricted flow passage of the SETA design will be easier to adapt for corrosion protection and sludge control than the EEI design. A more durable material will be required for the water header pipe than the mild steel used for prototype studies. The cost of a pavement heating system cannot be justified on the basis of eliminating routine winter maintenance operations. Other factors such as extended structure life and decreased costs passed on to the public and commercial transportation sector may justify pavement heating over fairly large areas. It is obvious that elimination of particular problem areas of localized extent may well be cost-beneficial and would warrant study on a case by case basis.

References

1. Swanson, Herbert N., "Evaluation of Geothermal Energy for Heating Highway Structures," Report CDOH-DTP-R-80-6 (1980).

2. Nydahl, J., Pell, K., et al., "Data Collection and Analysis for Geothermal Research," Report CDOH-UW-R-81-11 (1981).

3. Long, D.C. and Baldwin, J.S., "Snow and Ice Removal from Pavement Using Stored Earth Energy," FHWA-TS-80-227 (1980).

4. Wilson, C.H., et al., "A Demonstration Project for De-icing of Bridge Decks," Transportation Research Record 664 Bridge Engineering, Vol. 1 (1978), p. 189.

5. Pell, K., Nydahl, J.E. and Cundy, Vic, "Geothermal Heating of Bridge Decks," Snow Removal and Ice Control Research, Special Report 185, Transportation Research Board (1978), p. 169.

List of Tables

TABLE 1. MONTHLY AVERAGE ENVIRONMENTAL PARAMETERS

	Climatological Record (1931-1960)		Experimental Site Data (Winter 1980-1981)			
Month	Air Temp. (°C)	Precip. (cm)	Air Temp. (°C)	Precip.[a] (cm)	Wind Speed (m/s)	% Month Recorded
Sept	16.4	3.6	14.5	-	.9	37
Oct	10.6	3.6	9.1	-	.8	95
Nov	2.4	3.1	-	-	-	0
Dec	-2.6	3.7	1.4	4.6	1.0	71
Jan	-4.0	4.6	0.2	.5[b]	1.0	93
Feb	-1.3	4.5	-.2	2.2	1.2	83
Mar	3.3	3.9	4.9	8.3	2.1	38
Apr	8.7	4.8	11.1	6.2	2.1	98

[a] Monthly Values as Reported in the Glenwood Springs Newspaper

[b] The Major Snow Storm Was Not Recorded

TABLE 2. MONTHLY AVERAGED WATER TEMPERATURES AND FLOW RATES AND CORRESPONDING TEMPERATURE INCREASES OF HEATED SURFACES.

Water			Air	Heat Surfaces Temperature Increases (°C)				
Month	Temp (°C)	Flow (GPM)	Temp (°C)	SETA 6"	SETA 12"	SETA 18"	EEI 6"	EEI 8"
4/1980	26	136	3.4	7.6	6.4	5.3	4.7	3.4
9	29	-	14.5	4.5	3.3	3.1	4.8	2.4
10	38	33	9.1	4.8	3.8	3.3	5.8	3.2
12	25	32	1.4	6.3	6.4	5.5	9.3	5.5
1/1981	24	52	0.2	6.0	6.3	5.3	8.9	5.4
2	24	50	-0.2	7.3	6.7	6.5	9.6	6.0
3	26	98	4.9	5.6	3.7	3.6	4.6	2.4
4	27	92	11.1	4.0	3.3	3.2	4.0	1.9

TABLE 3. PERCENT REDUCTION IN VARIOUS SURFACE FREEZING CHARACTERISTICS OF THE HEATED SECTIONS

Heated Section	Time Frozen	°C Days below Freezing	Freeze-Thaw Cycles	Snow Cover Duration
SETA 6"	94	95	73	99
SETA 12"	91	95	74	99
SETA 18"	85	90	66	96
EEI 6"	99	99	95	100
EEI 8"	89	93	66	97

LIST OF FIGURES

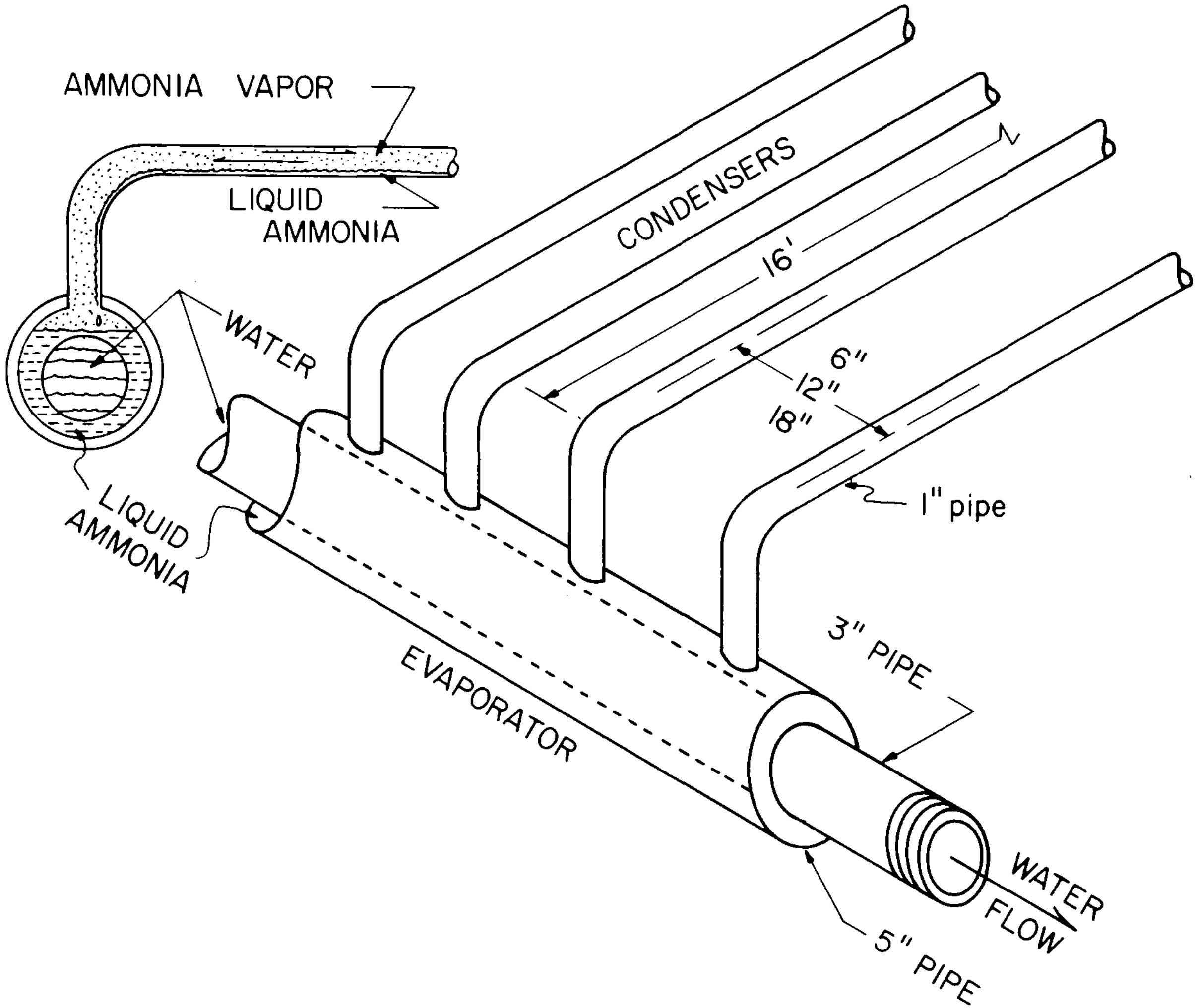


Figure 1. SETA's Heat Pipe Design for the Glenwood Springs Geothermally Heated Bridge.

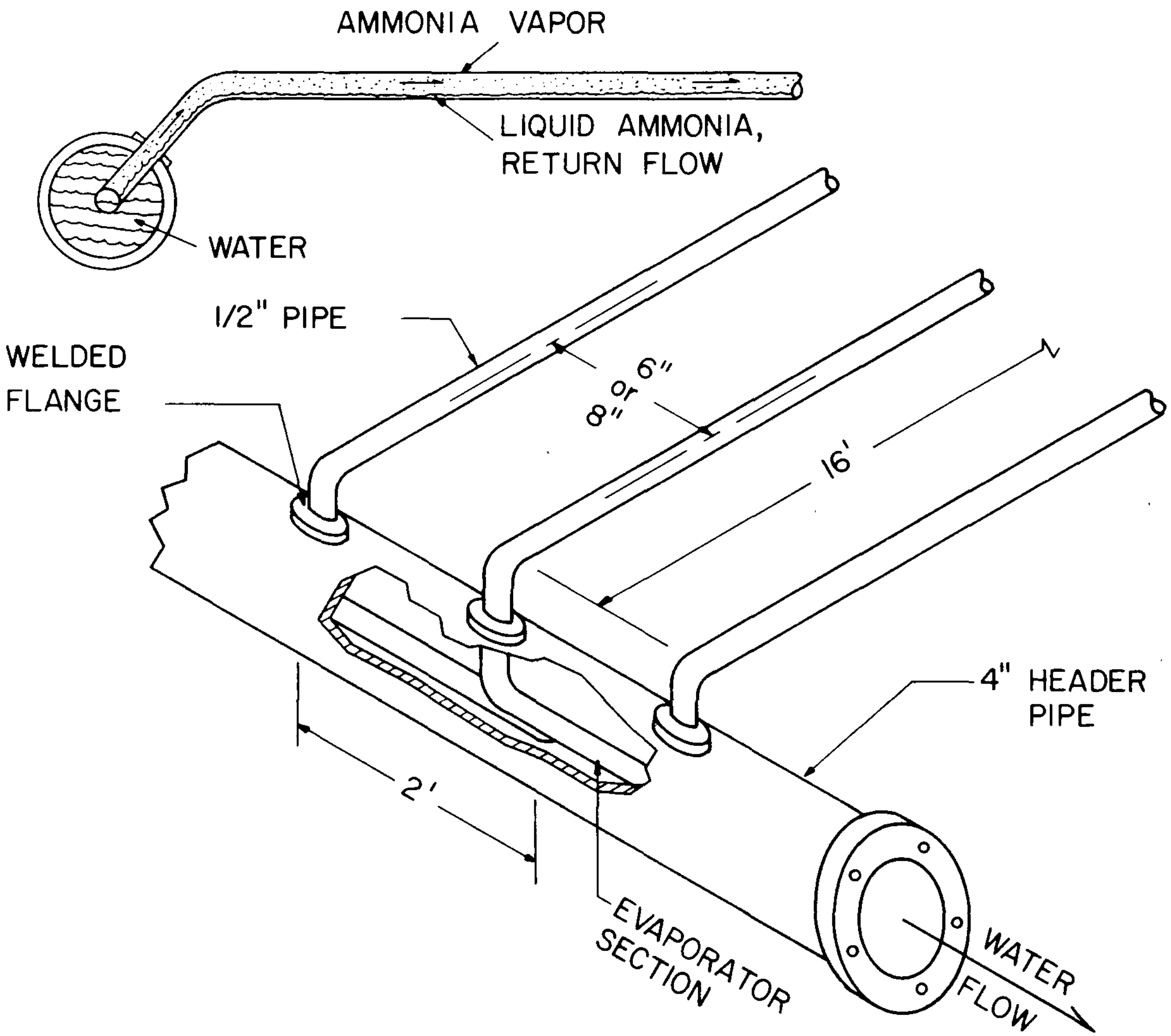


Figure 2. EEI's Heat Pipe Design for the Glenwood Springs Geothermally Heated Bridge.

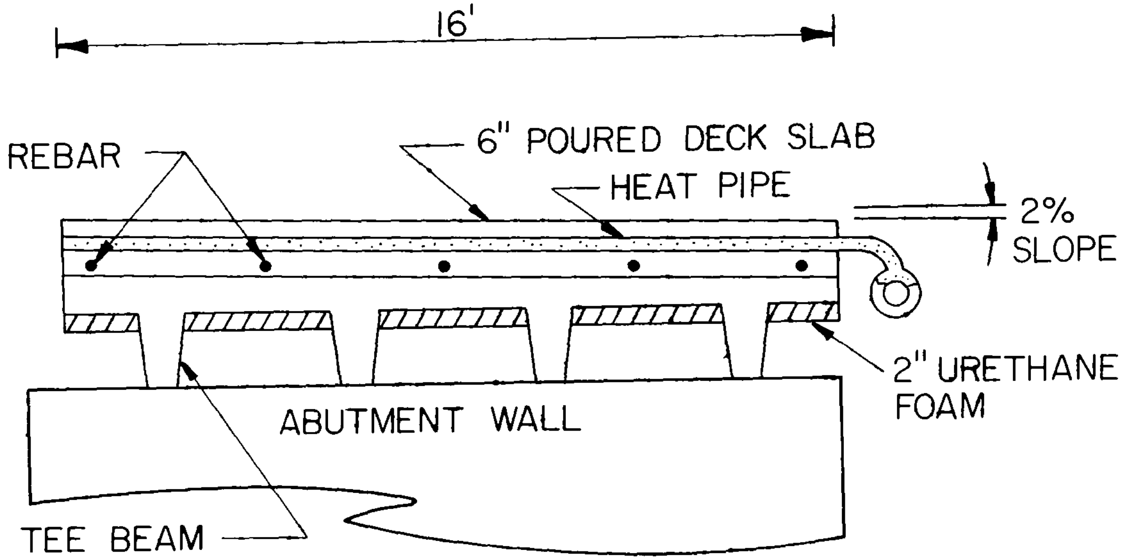


Figure 3. End View of Prototype Bridge.

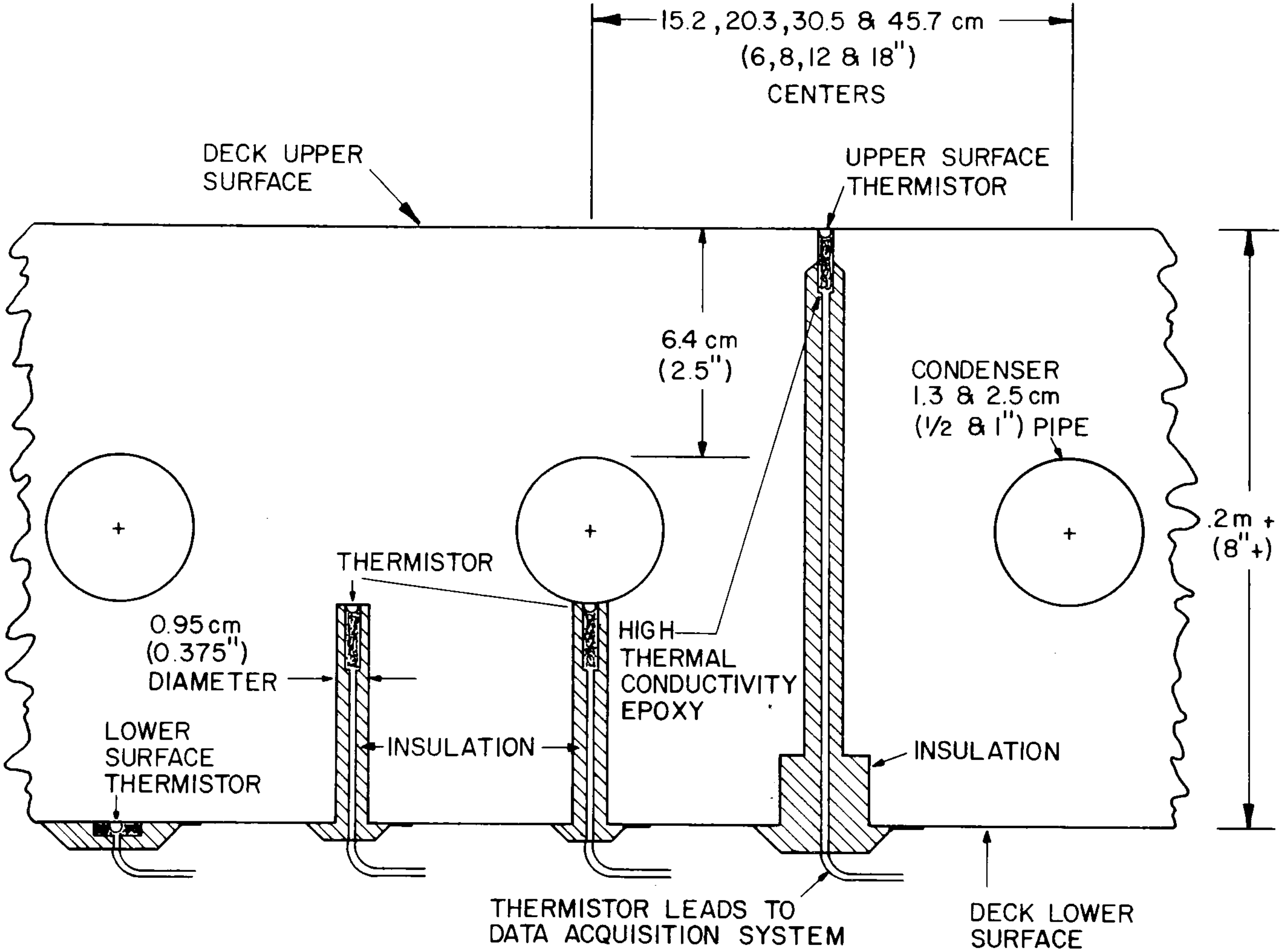


Figure 4. Typical Temperature Instrumentation in the Deck.

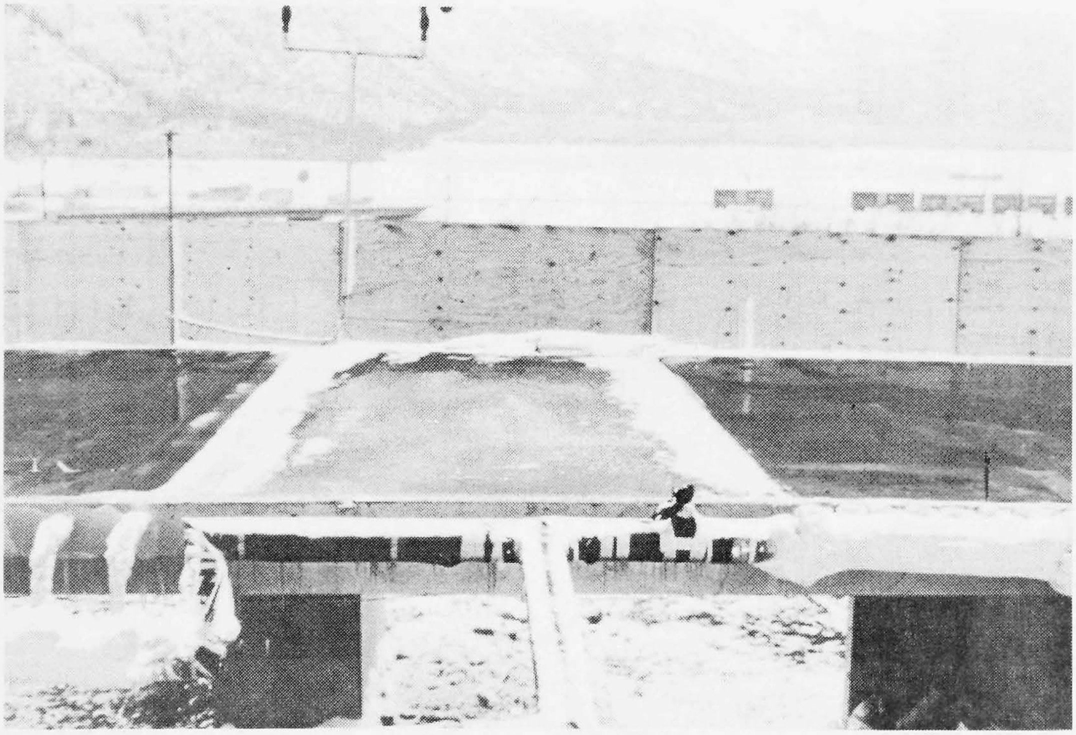


Figure 5. Typical Melting Event with an Ice Covered Unheated Control and Wet Heated Surfaces.

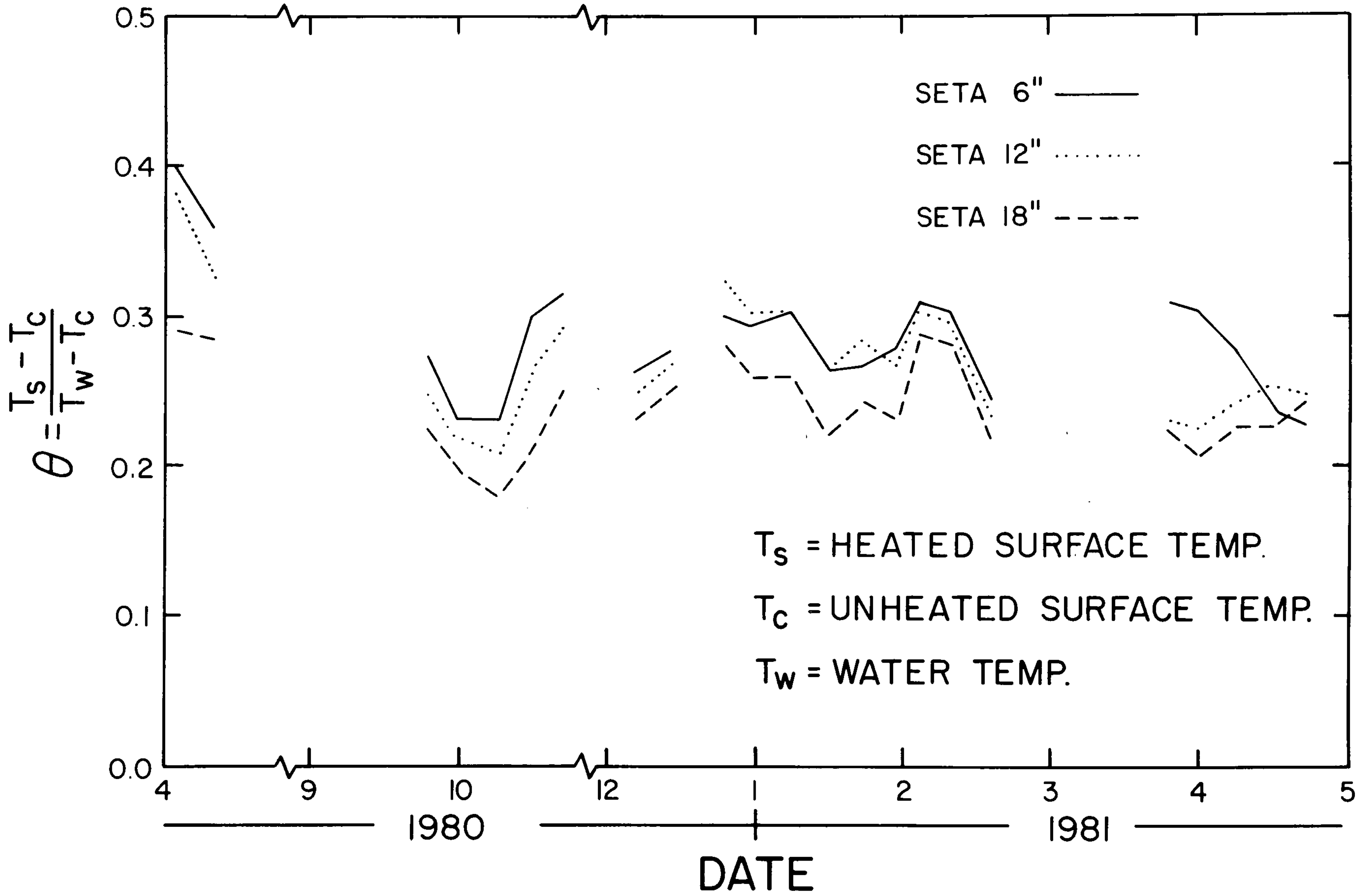


Figure 6. Coefficient of Performance θ versus Time for the SETA Heat Pipe Modulus.

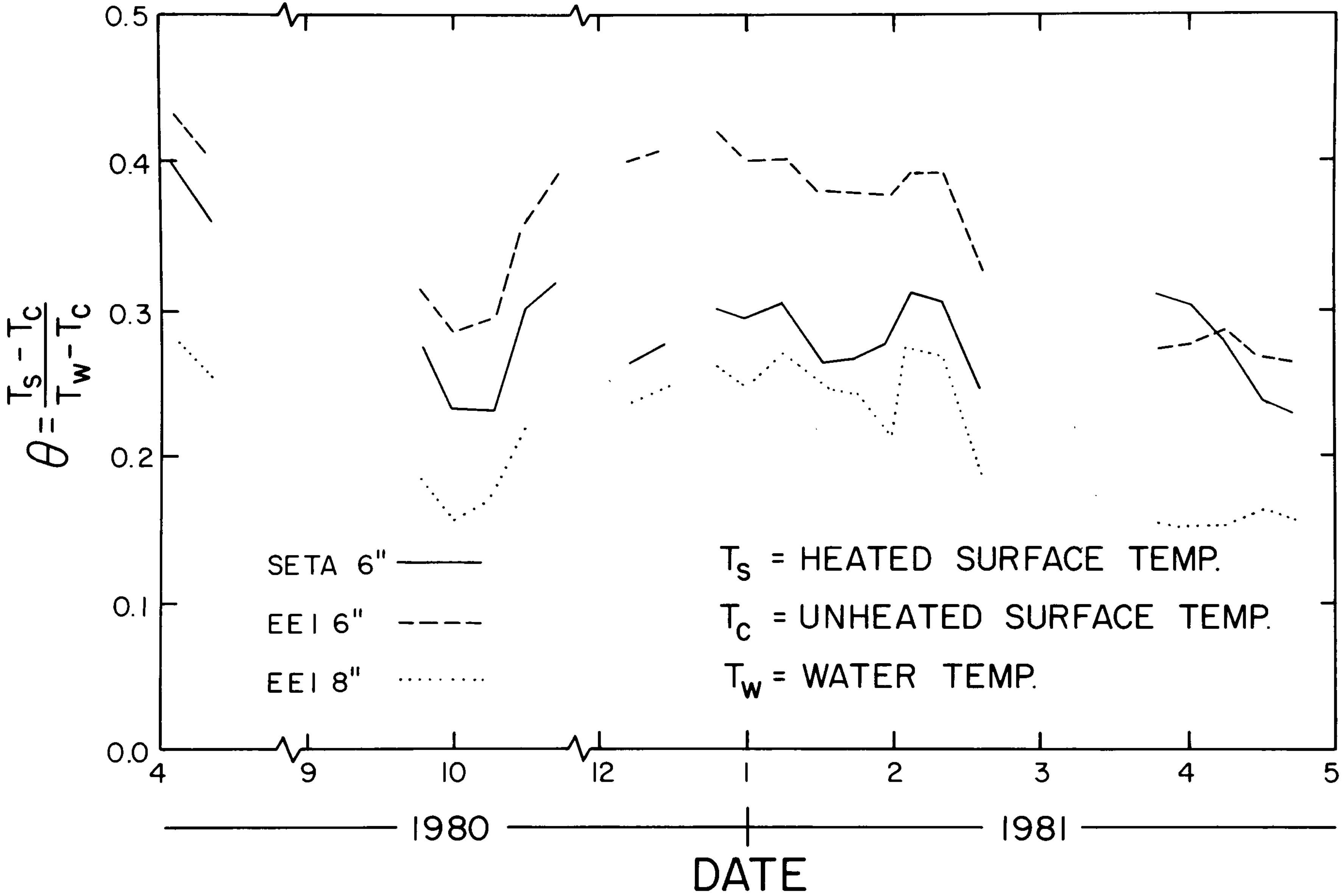


Figure 7. Coefficient of Performance θ versus Time for the EEI Heat Pipe Modulus and the SETA 6" Modulus.

ABSTRACT

Predicting the Strength of Field Compacted Soil from Laboratory Tests

by

Y. Liang and C.W. Lovell
School of Civil Engineering
Purdue University, W. Lafayette, Indiana

One approach to the determination of strength for field compacted soils is to construct test fills. After rolling at specified water contents with presumedly efficient rollers, the fill can be sampled, and the samples appropriately tested in the laboratory. In addition to increasing construction costs, this procedure stretches out the time during which cut and fill slopes are exposed and subject to erosion. This paper presents evidence that field compacted strength can be adequately predicted from tests on laboratory compacted samples. These samples can be acquired and tested in the preconstruction period, and the added cost and time delay associated with test pads can be avoided.

The test data presented are for both laboratory and field compaction of residual clays and involve: (a) confined undrained triaxial testing; (b) volume change produced by saturating the compacted samples with simulated embankment confinement, and (c) undrained triaxial testing of the saturated samples. Statistically valid regression equations were developed for the above responses in terms of variables such as dry density, water content, energy level, and confinement. These equations were found to have essentially the same functional form for both field and laboratory compacted soils, which makes it simpler to predict the field compacted response from testing of laboratory compacted samples.

INTRODUCTION

The effectiveness of compaction should be determined not on the basis of the density achieved but rather on the basis of the improvement of the specific properties pertinent to the particular application.

In the case of an embankment design, the engineer should select or estimate the expected soil strength, as well as develop specifications which insure the achievement of this strength. The most direct approach is to construct test pads using an appropriate range of compaction variables (moisture content, roller type and rating, roller use, etc.). The compacted soil is sampled and tested to determine the experimental relationships between the compaction process and the end results.

This procedure is both time consuming and expensive, and stretches out the time during which cut and fill slopes are exposed and subject to erosion. It can be avoided if field compacted properties are correlated with laboratory compacted ones.

This paper reports the experimental development of laboratory-field correlations for a midwestern residual clay from shale. Test pads were constructed, samples were taken, and tests were run which allowed evaluation of as-compacted strength, volume change when saturated under confinement, and parameters of saturated undrained strength. These dependent variables were related to the independent compaction ones in the form of statistically valid equations.

Relationships of the same type had previously been established for laboratory compacted samples from the same soil profile (Weitzel and Lovell, 1979; Johnson and Lovell, 1979; Weitzel and Lovell, 1980; and Lovell and Johnson, 1981).

Accordingly, it was possible to compare laboratory and field compacted strength relationships for similar soils, and to develop a strategy for predicting field compacted behavior from tests on laboratory compacted samples.

BACKGROUND

The general similarity of moisture-density relations between field and laboratory compaction (Johnson and Sallberg, 1960) is to be expected. Proctor developed the popular laboratory control methodology before much compaction equipment was available. Accordingly, modern compaction equipment has been developed to efficiently meet values of moisture-density produced in a laboratory test.

There is less evidence on the similarity of compacted properties such as strength. However, Holtz and Ellis (1964) found good correspondence between laboratory and field compacted strengths for a lean clay. Liang and Lovell (1982) have also reported similar strength functions for plastic residual clays, and these are described in detail later in this paper. Lin and Lovell (1981) show similarity in compressibility relationships for the same soils.

Other implicit evidence that laboratory and field compaction to a given moisture-density combination will produce very similar properties is afforded by practical experience. Engineers have tacitly assumed that laboratory compacted values are suitable estimates for field compacted soils for many years. The information which follows quantifies the laboratory-field correlation for a selected range of independent variables.

SOILS

The soils selected for study came from a profile developed on shale and sandstone in southern Indiana. The results of classification tests on both the laboratory and field compacted samples are given in Table 1. The material to be laboratory compacted was thoroughly mixed before testing and hence represents a single rather plastic soil. The field compacted test pad soils were mixtures of two strata, viz., an upper less plastic one and the deeper more plastic one used for the laboratory compaction studies. The field compacted samples had a considerable range of plasticities and mean values which were less than the laboratory samples.

TABLE 1. Index Properties and Classification of Test Clays.

		Test Pad	Laboratory
	w_L	40.0 (30.0, 53.2)	52
Atterberg Limits (%)	w_p	18.4 (16.7, 21.3)	23
Mean (low, high)	I_p	21.9 (16.4, 29.0)	29
Shrinkage Limit, (%)		11.8	12
Specific Gravity, G_s		2.79	2.80
Percent finer than 0.002 mm (Clay Content)		34	40
Skempton's Activity		0.65	0.73
Unified Soil Classification		CL	CH
AASHTO Classification		A-6	A-7-6

Laboratory impact type compaction curves are shown in Figure 1 for both a typical test pad soil and the laboratory study soil. The relative positions and shapes of these curves are as would be predicted from their relative plasticities. Comparing the Standard Proctor energy levels (2, Figure 1), the less plastic test pad soil has a greater maximum density, a lesser optimum moisture content, and a sharper peak.

EXPERIMENTAL PROCEDURES

Laboratory samples were compacted to fit the impact curves shown in Figure 1. To best approximate field rolling, kneading compaction was used in the laboratory. This extra precaution may not be necessary, and is being re-examined in current research. Field compaction was by two roller types, viz., a Raygo Rascal model 420C vibratory drum compactor and a Caterpillar model 825 tamping foot roller.

Two series of strength tests were undertaken for both laboratory and field compacted soil samples: (1) unconsolidated undrained (UU) triaxial tests, and (2) isotropically consolidated undrained triaxial tests with pore pressure measurements ($\overline{CIU}$). The first set of tests defined the end-of-construction strength values. The second set was performed after back-pressure saturation, under confinement approximating various embankment positions, and represented the resistance to a long-term undrained shear failure. The volume change induced by the saturation became a dependent variable, as did the transient pore pressure at shear failure. All testing details are completely defined in Weitzel and Lovell (1979), Johnson and Lovell (1979) and Liang and Lovell (1982).

RESULTS

The development of statistically valid relationships between compaction variables and compacted shear behavior was a primary objective of this research. The regression analysis used to accomplish this objective, and the criteria applied for statistical acceptance, are given in detail in Liang and Lovell (1982). The process of selecting independent variables and their form is essentially one of many trials. The ultimate choices are based upon simplicity and best statistical credentials, principally the multiple coefficient of determination (R^2).

Prediction models are shown for: as-compacted strength; volume change caused by saturation under confinement; and Skempton's pore pressure parameter at failure (A_f). Effective stress parameters c' and ϕ' have also been determined for saturated compacted soils. However, the data are not totally consistent, and research continues with respect to these predictions.

As-Compacted Strength

Equations 1 and 2 are the best relationships developed for field compacted and laboratory compacted soils, respectively.

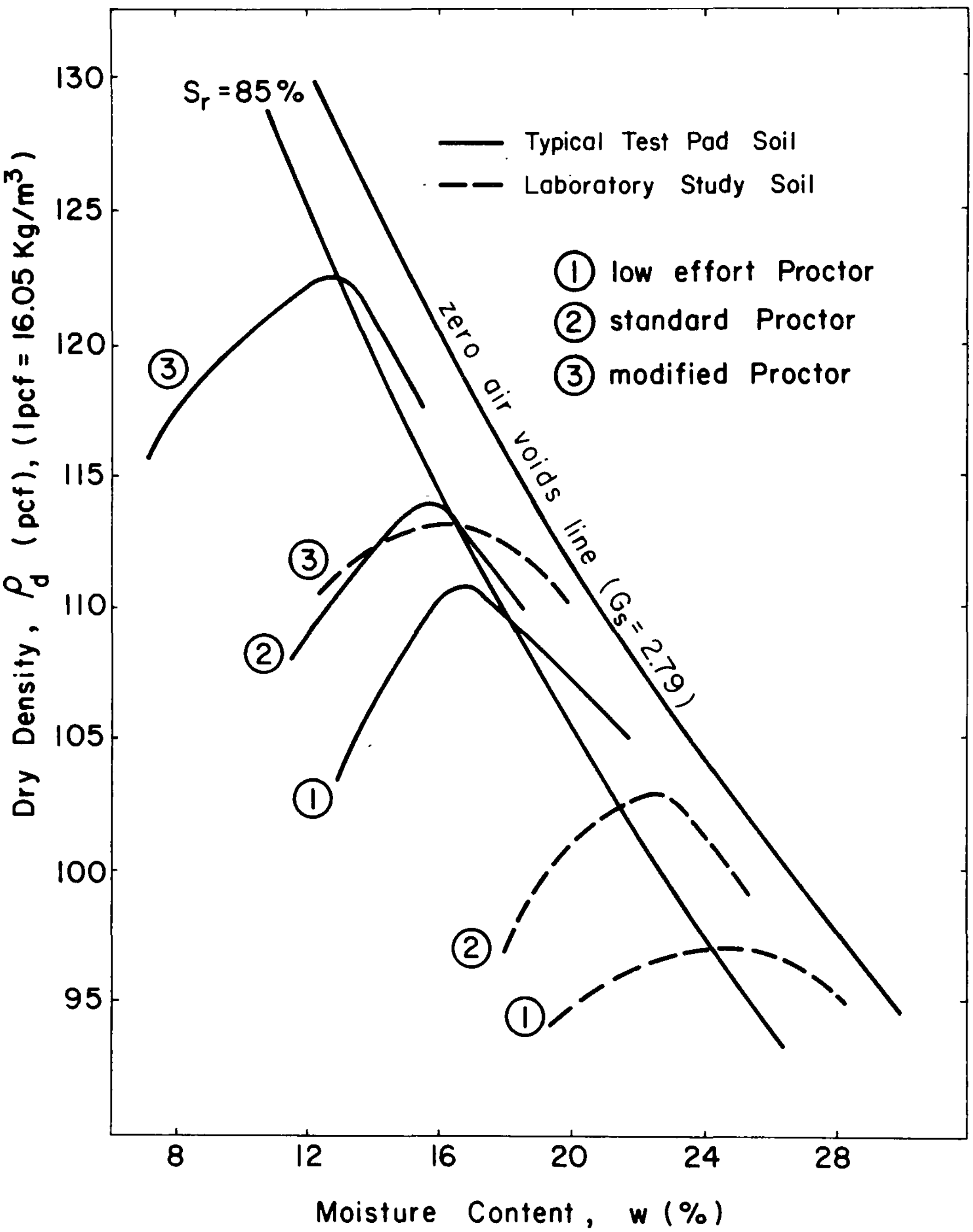


Figure 1: Impact Compaction Curves for Test Pad and Laboratory Study Soils.

(a) Field compacted:

$$\hat{q}_c = -6980.2 + 636.21\,w - 8.3\,w^2 - 0.155\,\rho_d\,w + 112.1\,(1 - S_i/100)\sqrt{\sigma_3} + 3.6\,\rho_d\sqrt{S_i/w} \quad \ldots (1)$$

(b) Laboratory compacted:

$$\hat{q}_c = -1878.2 + 51.54\,w + 1.39\,w^2 - 0.06\,\rho_d\,w + 76.9\,(1 - S_i/100)\sqrt{\sigma_3} + 3.68\,\rho_d\sqrt{S_i/w} \quad \ldots (2)$$

where $\hat{q}_c$ = estimated compressive strength, kPa
ρ_d = dry density, kg/m^3
S_i = initial degree of saturation, %
w = water content, %
σ_3 = confining pressure, kPa.

The R^2 value for the laboratory compacted samples is an excellent 0.98, while that for the field compacted is an adequate 0.72. The equations are complex, involving interactions and exponents, but the trends are clearly shown in Figure 2. The as-compacted strengths increase with increasing density and decreasing water content, for a given confining pressure, until the sample reaches near saturation at a high water content level. The laboratory and field models are quite similar.

Volume Change When Saturated Under Confinement

These measurements may be used to predict the settlements (heaves) which occur in service due to embankment saturation. Similar measurements may be made at a K_o ratio of stresses in the oedometer test (Lin and Lovell, 1981). The boundary stress ratio in the tests reported here is, of course, unity.

The volume change is strongly influenced by the confining pressure and the prestress value produced by compaction. This prestress is most directly determined in the oedometer test (DiBernardo and Lovell, 1979 and Lin and Lovell, 1981). The final regression models selected for field and laboratory soils are shown in equations 3 and 4.

(a) Field compacted:

$$\frac{\Delta\hat{V}}{V_o}\,(\%) = -0.166 + 2.47\,e_o\sqrt{\sigma'_c} - 0.365\,\hat{P}_s - 0.00263\,w\,\sigma'_c \quad \ldots (3)$$

(b) Laboratory compacted:

$$\frac{\Delta\hat{V}}{V_o}\,(\%) = -9.4 + 2.9\,e_o\sqrt{\sigma'_c} - 0.404\,\hat{P}_s - 0.00276\,w\,\sigma'_c \quad \ldots (4)$$

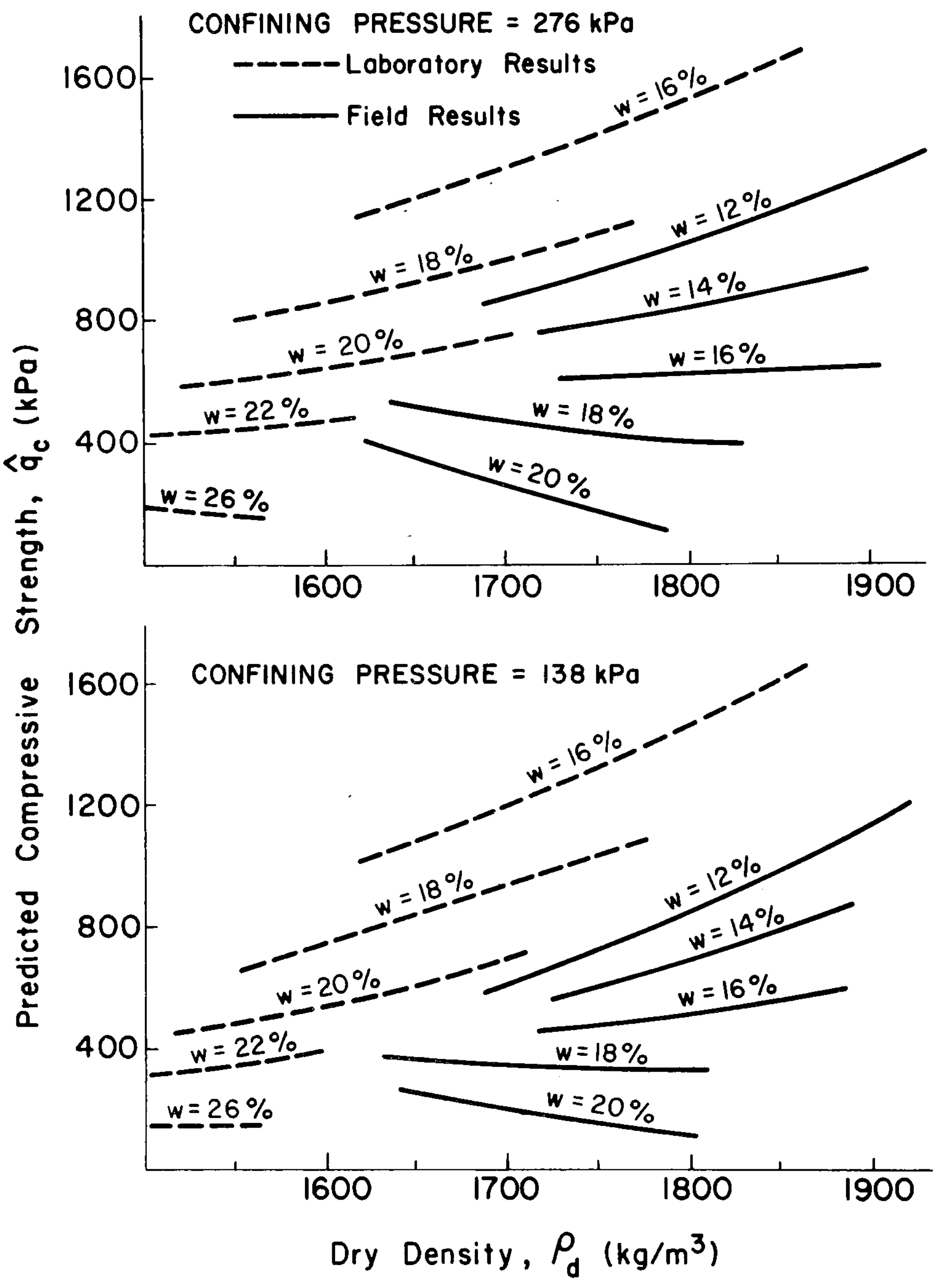


Figure 2: Predicted Compressive Strength–Dry Density Relationship at Constant Water Content.

where $\frac{\Delta\hat{V}}{V_o}$ (%) = estimated value of percent volume change due to saturation under confinement (swell is negative)

e_o = initial void ratio

σ_c' = isotropic consolidation pressure, kPa

w = initial water content, %

$\hat{P}_s$ = estimated compactive prestress, kPa.

The multiple coefficient of determination (R^2) for field compacted soils is 0.72, while that for laboratory compacted ones is 0.95. As illustrated in Figures 3 and 4, the field and laboratory relations are quite similar. Greater amounts of settlement ($+\frac{\Delta\hat{V}}{V_o}$) occur for lower values of compactive prestress, lower values of water content and higher values of confinement, for a given value of void ratio (or density).

Skempton's A_f Parameter

This ratio of pore pressure change induced by undrained failure to the boundary stress change producing the failure is used to characterize the effects of compaction variables on mobilization of effective stress in the saturated compacted soil. High positive values of A_f mean much reduced effective stress, and conversely negative values mean increased effective stress.

The parameter is a function of the overconsolidation ratio (Holtz and Kovacs, 1981). The best estimate of this ratio (OCR) for compacted soils in this study is the compactive prestress ($\hat{P}_s$) divided by the effective confining pressure (σ_c'), i.e., OCR = $\hat{P}_s/\sigma_c'$. Note that $\hat{P}_s$ for the as-compactive soil is estimated as only a total stress, not as an effective stress.

The final regression models selected for the field compacted and laboratory compacted soils are given by equations 5 and 6, respectively.

(a) Field compacted:

$$\hat{A}_f = 2.05 - 0.73/e_o - 0.232 \times 10^{-4} \rho_d \sqrt{S_i} - 0.382 \log(\text{OCR}) \quad \ldots (5)$$

(b) Laboratory compacted:

$$\hat{A}_f = 2.34 + 0.56/e_o - 0.189 \times 10^{-3} \rho_d \sqrt{S_i} - 0.246 \log(\text{OCR}) \quad \ldots (6)$$

where e_o = initial void ratio

ρ_d = initial dry density, kg/m^3

S_i = initial degree of saturation, %

OCR = the ratio of estimated compactive prestress ($\hat{P}_s$) to the isotropic effective confining pressure (σ_c').

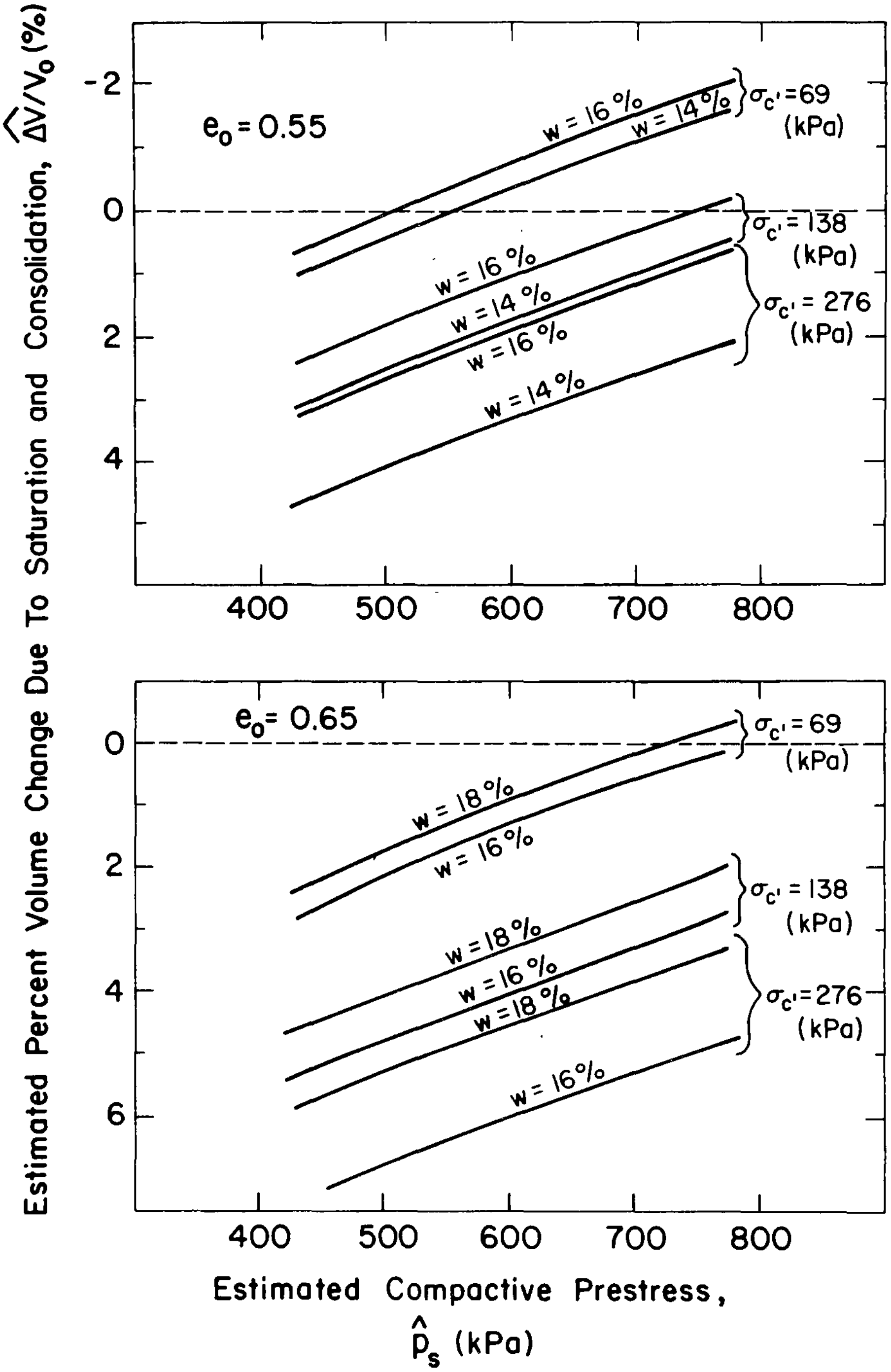


Figure 3: Prediction of Field Percent Volume Change Due to Saturation Under Confinement at a Constant Initial Void Ratio. (Swell is Negative)

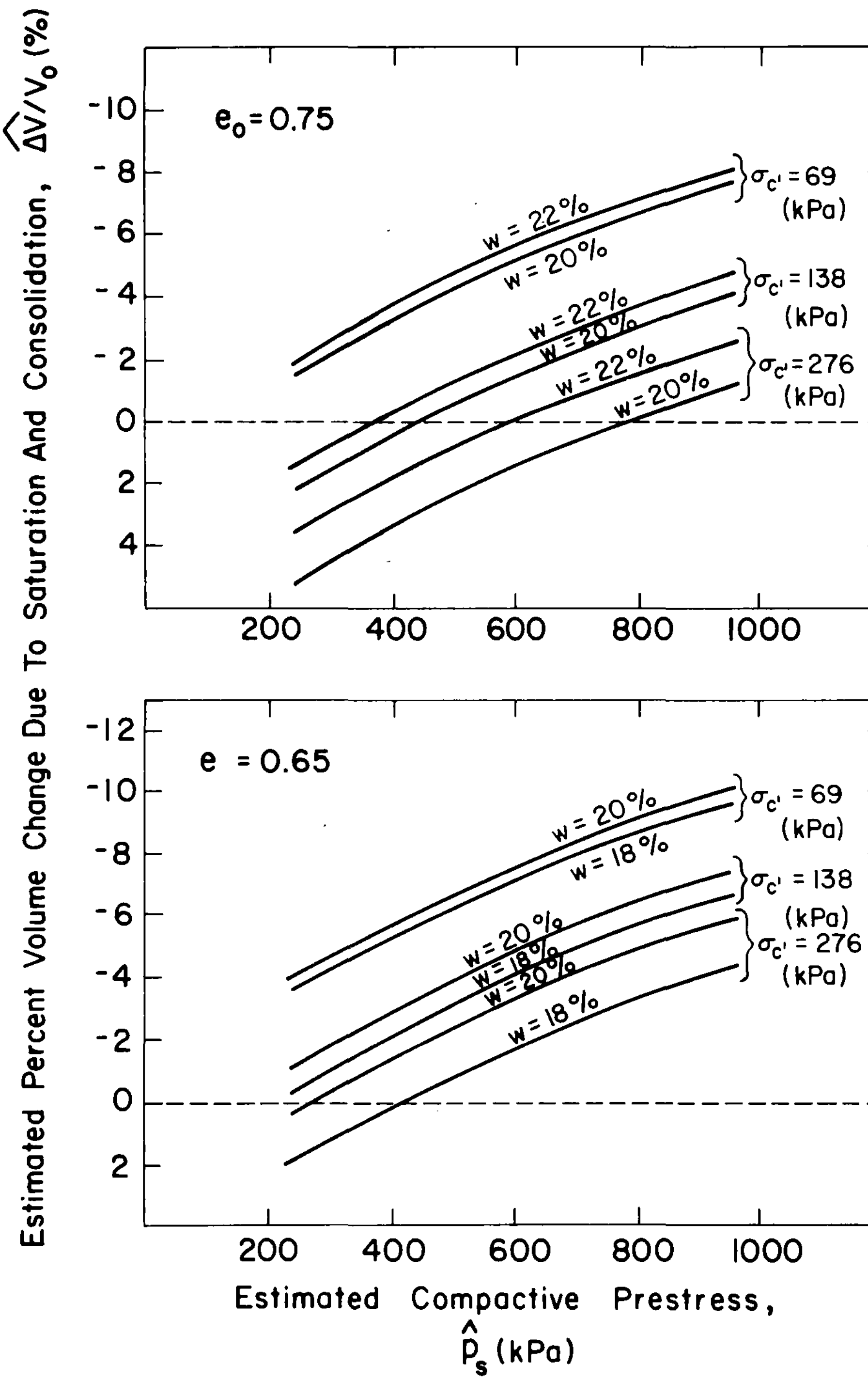


Figure 4: Prediction of Laboratory Percent Volume Change Due to Saturation Under Confinement at a Constant Initial Void Ratio. (Swell is Negative)

The multiple coefficients of determination (R^2) for field and laboratory compacted soils are 0.63 and 0.72, respectively. These values are low but statistically acceptible.

The essential trends are illustrated in Figures 5 and 6. The A_f values are lower (effective stresses at failure are higher) as the OCR increases and the initial void ratio decreases (initial density increases). The trends shown are for a constant initial value of degree of saturation. They are similar, but the initial void ratio has a greater effect for field compacted than for laboratory compacted.

PREDICTION OF FIELD COMPACTED RELATIONSHIPS FROM LABORATORY TESTS

Where the appropriate functional relations have been directly generated in the field and in the laboratory, and for the same soil, the evidence presented above shows that the prediction should be very simple. It would be best accomplished by plotting and interpreting figures like 2 through 6.

Not all combinations of variables can be experimentally covered. However, when a number of them have, the engineer will be able to make comfortable interpolations among the data available.

In many practical cases the laboratory data will be available for a soil which is somewhat different than the soil to be used in the real embankment...this is the nature of the experimental data which has been presented in this paper. To resolve the problem, a conversion factor may be applied, which adjusts for the soil differences. Lin and Lovell (1981) proposed that the ratio of Atterberg plasticity indexes for the two soils be used for this purpose. With reference to Figure 1, this approach would use the laboratory curve 2 (dashed line) to predict the field performance of a similar soil represented by curve 2 (solid line).

The approach reported by Liang and Lovell (1982), and recommended in this paper, relates the field performance to the laboratory curve for that same soil, i.e., curve 2 (solid line) is used directly. Curve 2 (solid line) may be experimentally generated for the field soil, or may be estimated by a family-of-curves approach (Liang and Lovell, 1982). The latter method is termed a "translated laboratory curve technique".

The laboratory compacted functional relationships are assumed to be valid for both soils, and the numerical values of independent variables for the field compacted soil are substituted in the prediction equations. After statistical analysis, the following prediction equations are produced.

(a) As-compacted strength:

$$\hat{q}_c \text{ (field)} = 174.92 + 0.916\ \hat{q}_c \text{ (laboratory)}$$
$$(R^2 = 0.84) \qquad \ldots (7)$$

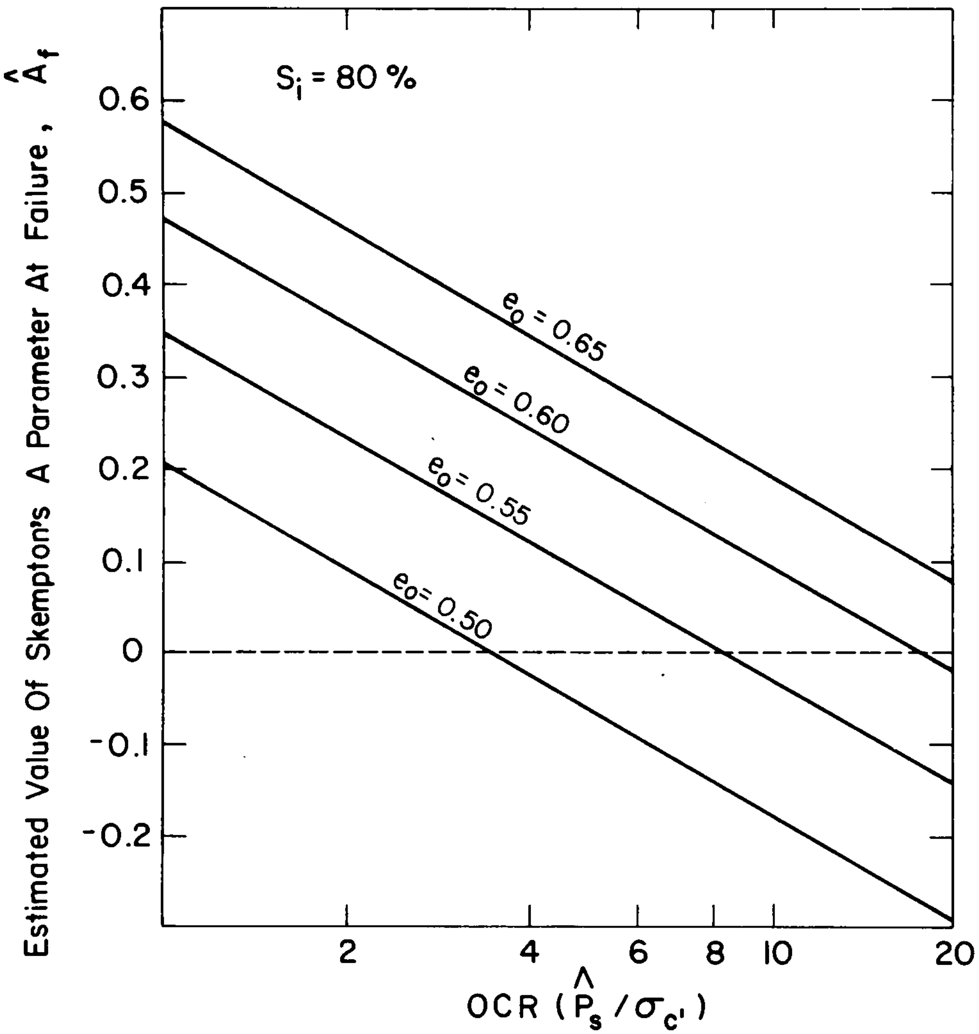


Figure 5: Field Skempton's A Parameter at Failure Verses OCR at a Constant Initial Saturation.

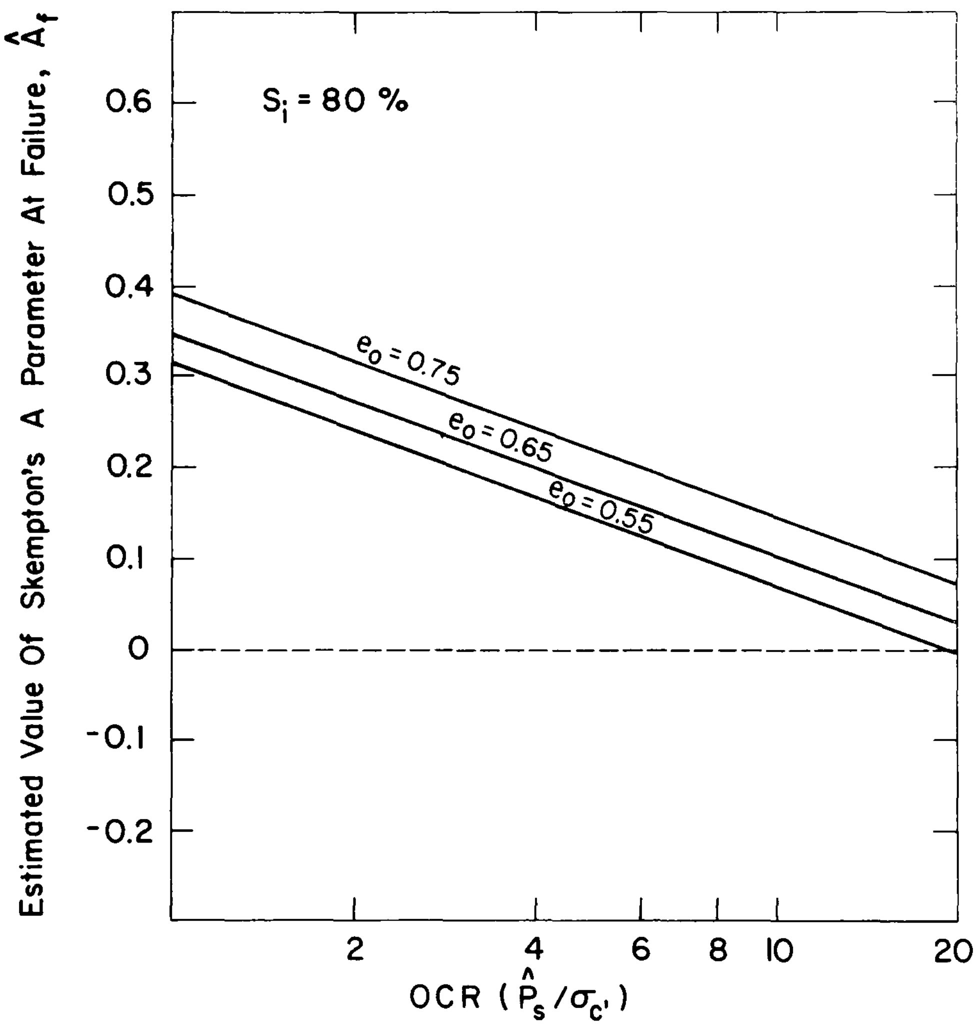


Figure 6: Laboratory Skempton's A Parameter at Failure Verses OCR at a Constant Initial Saturation.

where $\hat{q}_c$ is the estimated compressive strength in kPa. Typical values are shown in Figure 2.

(b) Volumetric strain due to saturation under confinement:

$$\frac{\Delta\hat{V}}{V_o}\text{ (field)} = 1.886 + 0.495\,\frac{\Delta\hat{V}}{V_o}\text{ (laboratory)}$$
$$(R^2 = 0.81) \qquad \ldots (8)$$

where $\frac{\Delta\hat{V}}{V_o}$ is the estimated volumetric strain as a fraction. Typical values (as percentages) are shown in Figure 3 for field compacted and in Figure 4 for laboratory compacted. Swell is negative.

(c) Skempton's A factor at failure:

Since this function contains the relation $\left(OCR = \frac{\hat{P}_s}{\bar{\sigma}_c'}\right)$, it was necessary to first relate field compacted and translated laboratory compacted values of prestress.

$$\hat{P}_s\text{ (field)} = -\ 303.67 + 1.323\ \hat{P}_s\text{ (laboratory)}$$
$$(R^2 = 0.91) \qquad \ldots (9)$$

where $\hat{P}_s$ is the estimated prestress value in kPa. See Figure 3 and 4 for typical values.

$$\hat{A}_f\text{ (field)} = 0.223 + 0.66\ \hat{A}_f\text{ (laboratory)}$$
$$(R^2 = 0.69) \qquad \ldots (10)$$

See Figure 5 for field compacted values and Figure 6 for laboratory compacted ones.

SUMMARY

Statistical regression equations have been presented for a family of field compacted clays. These included: as-compacted compressive strength, volume change when saturated under confinement, and Skempton's pore pressure parameter A_f from $CI\bar{U}$ testing. These experimental functions were found to be quite similar to those developed earlier by laboratory compaction for a more plastic clay from the same soil profile.

Assuming that the laboratory compaction functions for the more plastic soil were correct for the less plastic ones, the laboratory data were translated to values appropriate for the latter soils. Statistical equations were then developed which would allow the prediction of values for the dependent variables of field compacted soils from measurements made on similar laboratory compacted ones.

Although any conclusions to be drawn from the work are limited to the specific data fields studied, correlation of laboratory and field compacted properties seems direct and simple. With a limited number of studies of this type, it should be possible to adequately predict field compacted response from laboratory measurements, ultimately avoiding the need for test pads.

ACKNOWLEDGEMENTS

This study was sponsored by the Federal Highway Administration and the Indiana Department of Highways. It was administered through the Joint Highway Research Project, School of Civil Engineering, Purdue University, W. Lafayette, Indiana.

REFERENCES

DiBernardo, A. and Lovell, C. W. (1979), "The Effect of Laboratory Compaction on the Compressibility of a Compacted Highly Plastic Clay", Joint Highway Research Project Report No. 79-3, Purdue University, West Lafayette, Indiana, May, 187 pp.

Holtz, W. G. and Ellis, W. (1964), "Comparison of the Shear Strength of Laboratory and Field Compacted Soils", Laboratory Shear Testing of Soils, STP No. 361, ASTM, pp. 280-291.

Holtz, R. D. and Kovacs, W. D. (1981), "An Introduction to Geotechnical Engineering", Prentice-Hall, Inc., Englewood Cliffs, New Jersey, 733 pp.

Johnson, A. W. and Sallberg, J. R. (1960), "Factors that Influence Compaction of Soils and Compaction Characteristics of Field Equipment", Highway Research Board, Bulletin 272, Washington, D.C., 206 pp.

Johnson, J. M. and Lovell, C. W. (1979), "The Effect of Laboratory Compaction on the Shear Behavior of a Highly Plastic Clay After Saturation and Consolidation", Joint Highway Research Project Report No. 79-7, Purdue University, West Lafayette, Indiana, Aug., 271 pp.

Liang, Y. and Lovell, C. W. (1982), "Strength of Field Compaction Clayey Embankments", Joint Highway Research Project Report No. 82-1, Purdue University, West Lafayette, Indiana, Jan., 313 pp.

Lin, P. S. and Lovell, C. W. (1981), "Compressibility of Field Compacted Clay", Joint Highway Research Project Report No. 81-14, Purdue University, West Lafayette, Indiana, August, 154 pp.

Lovell, C. W. and Johnson, J. M. (1981), "Shearing Behavior of Compacted Clay After Saturation", Laboratory Shear Strength of Soil, ASTM, STP 740, Sept., pp. 277-293.

Weitzel, D. W. and Lovell, C. W. (1979), "The Effect of Laboratory Compaction on the Unconsolidation-Undrained Strength Behavior of a Highly Plastic Clay", Joint Highway Research Project Report No. 79-11, Purdue University, West Lafayette, Indiana, Sept., 218 pp.

Weitzel, D. W. and Lovell, C. W. (1980), "Prediction of Density and Strength for a Laboratory-Compacted Clay", Transportation Research Record 754, TRB, Nov., pp. 53-59.

DETERMINATION OF FRICTION ANGLE VALUES FOR ROCK DISCONTINUITIES IN REGARD TO STABILITY OF HIGHWAY CUTS

TEME S.C. and WEST T.R.*

Department of Geosciences
Purdue University
West Lafayette, Indiana 47907

Abstract

Stability of rock slopes depends, to a great extent, on the occurrence, distribution, orientation and the frictional properties of planes of weakness in the rock mass. Careful design of highway cuts and open pit mines in rock commonly includes detailed rock sampling and laboratory testing to determine the shear strength parameters (ϕ and C_d) of major rock discontinuity surfaces present in these slopes. Measured values of cohesion (C_d) and frictional angle (ϕ) are subsequently used to evaluate the potential stability of rock slopes based on the observed mode(s) of failure.

A laboratory method for testing shear strengths of rock discontinuities, using a shear box specially built at Purdue University, is discussed including a summary of the test results. Several sets of data have indicated its suitability as a routine procedure for rock slope design.

A possible range of applications of the shear box equipment involving several types of discontinuities is also presented. Advantages and limitations of the testing technique are also presented and discussed.

INTRODUCTION

Most rock slides that occur in the field are known to fail in shear, hence the shear strength of rock is an important design criterion for slope

---

* Graduate student and Associate Professor, respectively.

stability evaluations (6)*. Commonly the parameters of importance in the determination of the shear strength of rock are the friction angle (ϕ) and the cohesion intercept (C_d). However, the behavior of a rock mass is affected by the combined behavior of the continuous (or intact) rock and the discontinuities. In fact, the strength of a rock is determined by its weakness planes so that the occurrence, distribution, orientation and the frictional properties of these weakness planes govern the stability or instability of any given rock slope.

This paper defines and describes discontinuities and outlines a laboratory method for the determination of the shear strength parameters (ϕ and C_d) for the common-occurring discontinuity planes associated with routine rock mass sampling.

SIMPLE GEOMETRIES OF TYPICAL DISCONTINUITY PLANES ON A HIGHWAY ROCK SLOPE

The authors appreciate that clear definitions for terms related to discontinuity planes in a rock mass help provide a better understanding of the behavior and strength properties for discontinuities in general. In this respect, the discussion on discontinuities that follows relates to individual as well as groups of discontinuities in any given rock mass.

Basically, discontinuities are planar features (or surfaces) with a smaller resistance and higher deformability than the intact rock (3). By definition, an Individual Discontinuity is a planar surface in a rock mass, that separates two opposite portions of the wall rock. Special features that are usually associated with an individual discontinuity plane are (i) aperture, (ii) filler and (iii) asperities.

* Numbers in ( ) refer to the list of cited references at the end of this paper.

(i) An aperture is the distance between discontinuity surfaces, measured perpendicularly to the mean plane. This term, however, becomes rather ambiguous if the surfaces touch only at a few points such as shown in Figure 1.

(ii) The filler is the transported material which takes up the space between the discontinuity surfaces, either in a complete or partial fashion.

(iii) The asperities are the deviations of the surface of the discontinuity from a mean plane as viewed at various scales. The largest deviations are referred to as "undulations" whereas the smallest deviations are regarded as "roughness". Detailed descriptions of discontinuities in rock can be found in (1) and (2).

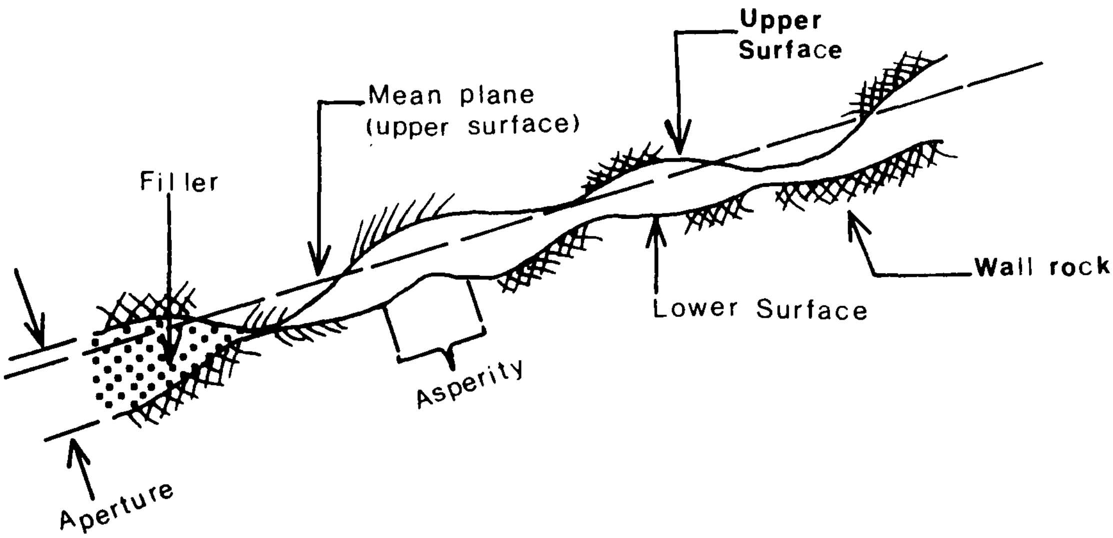


Figure 1. Characteristic features of an Individual Discontinuity (modified from 3)

Rock mass discontinuities are groups of planar surfaces that occur in a given rock mass. Characteristic features of rock mass discontinuities include (i) attitude, (ii) set, (iii) spacing, (iv) system, and (v) persistence. These features are shown schematically in Figure 2 and are defined below. These definitions are based on reference (7).

(i) Attitude of rock mass discontinuities is the orientation (i.e. strike and dip) of the discontinuity planes in space.

(ii) Discontinuity set is a group of discontinuities at various locations in the rock mass that are roughly parallel to each other.

(iii) Spacing is the distance between two adjacent discontinuities within a set.

(iv) Discontinuity system is a group of several sets.

(v) Persistence is the proportion of continuity along the mean plane. To illustrate, if the total length of a given discontinuity plane (including intact portions along the plane) is designated by A_{TOTAL}; and the total length of intact portions along the plane is A_{INTACT} then, by definition the persistence is $(A_{TOTAL}-A_{INTACT})/A_{TOTAL}$ (see Figure 2).

GEOLOGIC SETTING: TEST SAMPLES

As the values for the shear strength parameters (ϕ and C_d) for discontinuity surfaces of a given rock mass can be used to evaluate the stability of both highway and open pit mines, if they occur in the same geologic environment, the strength values obtained for the rock type described in this paper are equally applicable to both open pit and highway cut slopes in that similar environment.

In this study the test samples consisted of fine-grained sandstone collected from Greene and Sullivan counties in southwestern Indiana. This

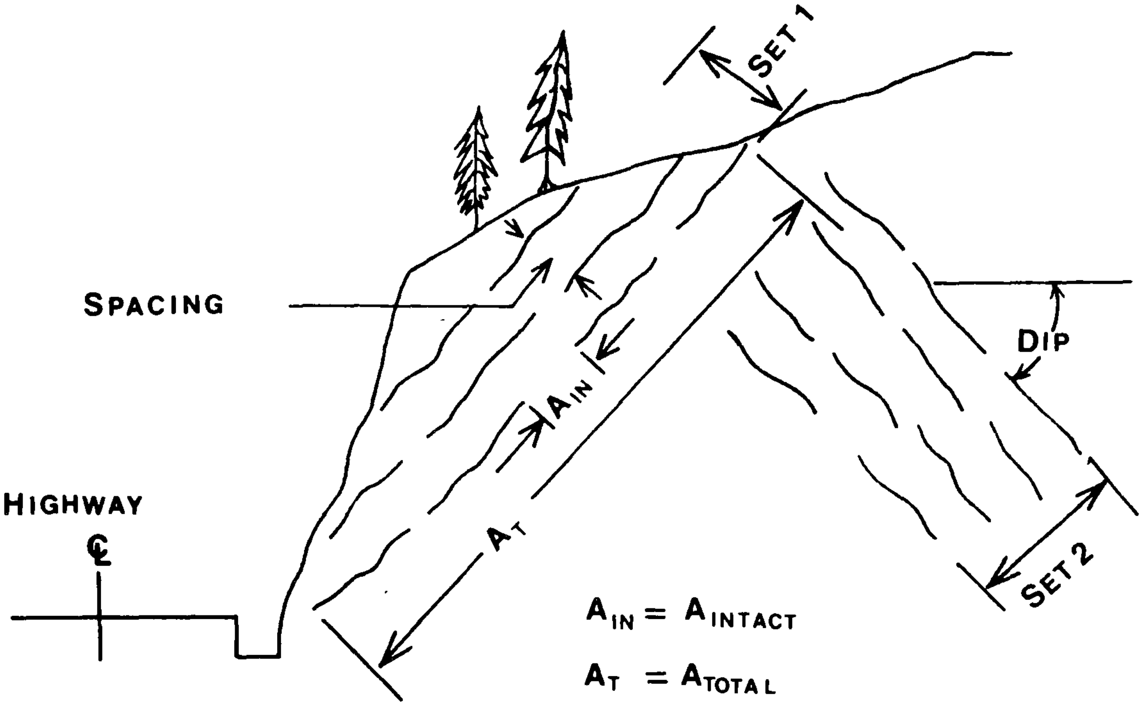


Figure 2. Geometry of typical discontinuity planes on a highway slope (modified from 3)

fairly-well indurated, fine-grained sandstone is from an unnamed member of the Dugger Formation, which is exposed about two miles northeast of the town of Dugger in Sullivan County, Indiana. The sites from which the samples were obtained are shown in Figure 3.

Stratigraphically, the Dugger Formation which ranges from 73 to 185 feet thick, comprises the uppermost formation in the Carbondale Group of the Pennsylvanian System in Indiana (10 and 11). This stratigraphic relationship is shown in part in the geologic record in Table 1. The sampled sandstone is the unnamed member above the Hymera Coal Member but below the Universal Limestone Member as shown in Table 1.

METHOD OF SAMPLING

The sampling method involved the dislodgement of large pieces from rock outcrops using a geological hammer. This procedure was such that most rock pieces typically contained noticeable discontinuity planes within them. In this situation, care was taken to maintain field discontinuity contacts until the time of testing when samples were prepared according to the procedure outlined below.

SAMPLE PREPARATION

Each sample for testing was cut using a band saw to yield dimensions approximately 2 inches long by 2 inches wide. The thickness of each specimen was governed by the undulation of the discontinuity plane it contained. Each half of the test specimen separated by a discontinuity plane (if any) was usually about 2 1/2 inches thick. For sawn test specimens (those without noticeable discontinuity planes), the thickness of each sample half was approximately 2 inches or slightly less.

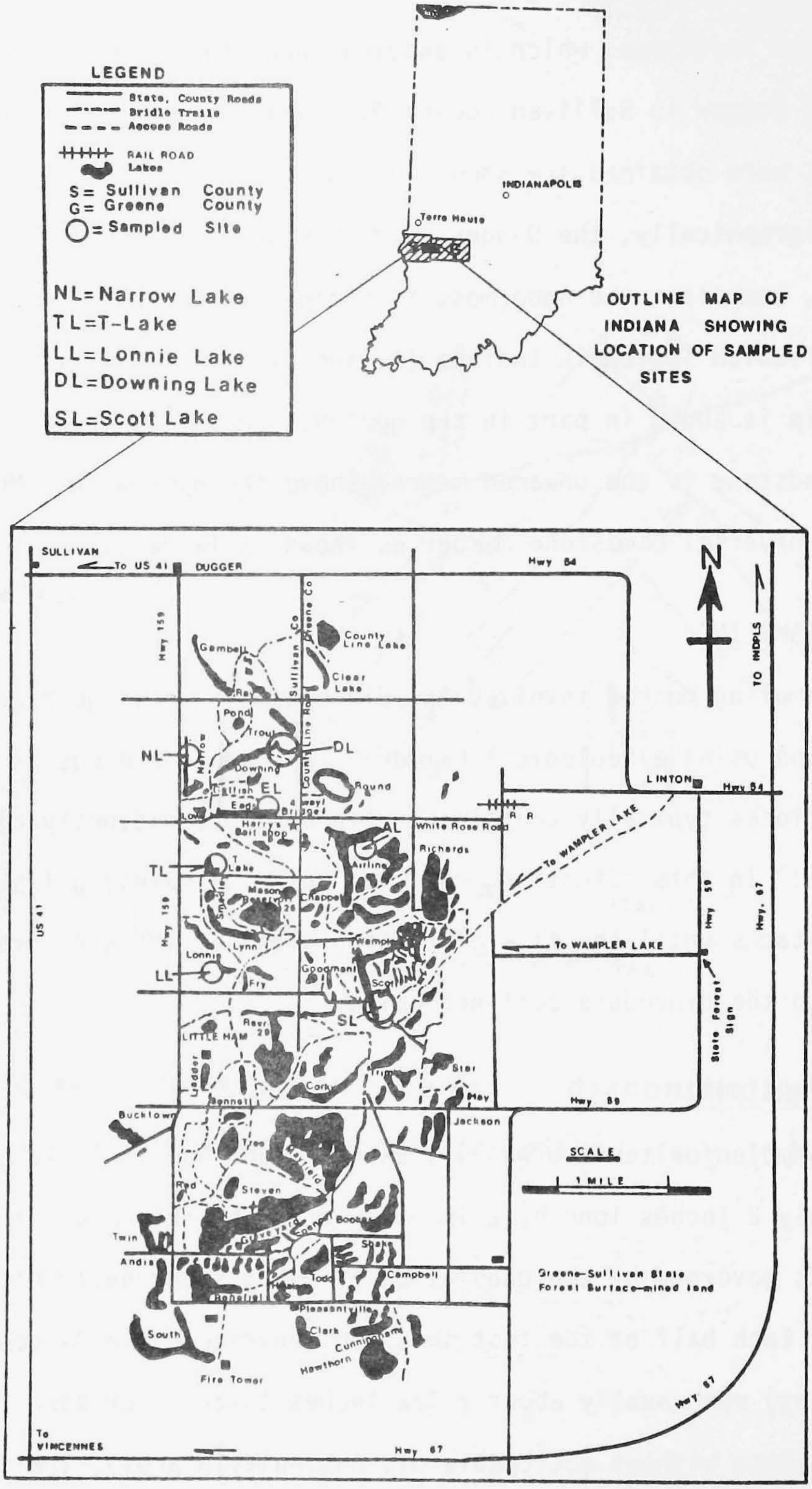


Figure 3. Locations of Sampled Sites for Laboratory Testing.

Table 1. Some Pennsylvanian Rock Units in Indiana (after Shaver, R.H. et al., 1970). ★indicates stratigraphic sequence of test samples.

SYSTEM	SERIES: APPALACHIAN	SERIES: MID-CONTINENT	GROUP	FORMATION	MEMBER AND BED
PENNSYLVANIAN	Conemaughian	Missourian	Mcleansboro		
		? Desmoinesian		Shelburn Fm	West Franklin Ls Mbr.
					Pirtle Coal Mbr.
					Busseron Ss Mbr.
	Alleghenian		Carbondale	Dugger Fm	Danville Coal Mbr (VII)
				★	Universal Ls Mbr ★
					Hymera Coal Mbr (VI)
					Providence Ls Mbr
					Herrin Coal Mbr
					Bucktown Coal Mbr (Vb)
					Antioch Ls Mbr
					Alum Cave Ls Mbr
				Petersburg Fm	Springfield Coal Mbr (V)
					Stendal Ls Mbr
					Houchin Creek Coal Mbr (IVa)
				Linton Fm	Survant Coal Mbr (IV)
					Velpen Ls Mbr
					Colchester Coal Mbr (IIIa)
					Coxville Ss Mbr
			Raccoon Creek	Staunton Fm	Seelyville Coal Mbr (III)
					Silverwood Ls Mbr
					Holland Ls Mbr
	Pottsvillian				Perth Ls Mbr
		? Atokan		Brazil Fm	Minshall & Buffaloville Coal Mbrs
					Upper Block Coal Mbr
					Lower Block Coal Mbr

Each half of the jointed (or sawn) sandstone specimen was set in hydrostone within two separate hollow cylindrical containers. Care was taken during this procedure that both halves of the jointed (or sawn) sandstone specimen fitted perfectly together when the two cylindrical containers were properly aligned.

TEST EQUIPMENT AND METHODOLOGY

The schematic diagram for the direct shear testing equipment is shown in Figure 4. Essentially it consists of movable and stationary boxes.

Inside these boxes are the hollow cylindrical sample containers which house the test samples during routine testing. The two boxes are held in contact by a loading frame through which the normal load is also applied. A labeled photograph of the direct shear box is shown in Figure 5.

Using the apparatus shown in Figures 4 and 5, it was possible to follow a loading path in which the normal stiffness was held constant. That is, the normal force was set at an initial value using a spring and the shear force was gradually increased until constant readings or a drop in readings were obtained in the shear dial gauges. The change in the normal force either during a test or during an increment at the end of a test, was equal to the displacement in the normal direction multiplied by a spring constant.

This shear apparatus is generally similar to the Ohio River District (U.S. Army Corps of Engineers) shear apparatus described in (3) except that the normal load in this case is applied through 20 Belleville Spring washers arranged in series instead of using a hydraulic jack. The deflections produced in the normal displacement dial gauges, as a result of compression of the washers, is multiplied by the spring constant to

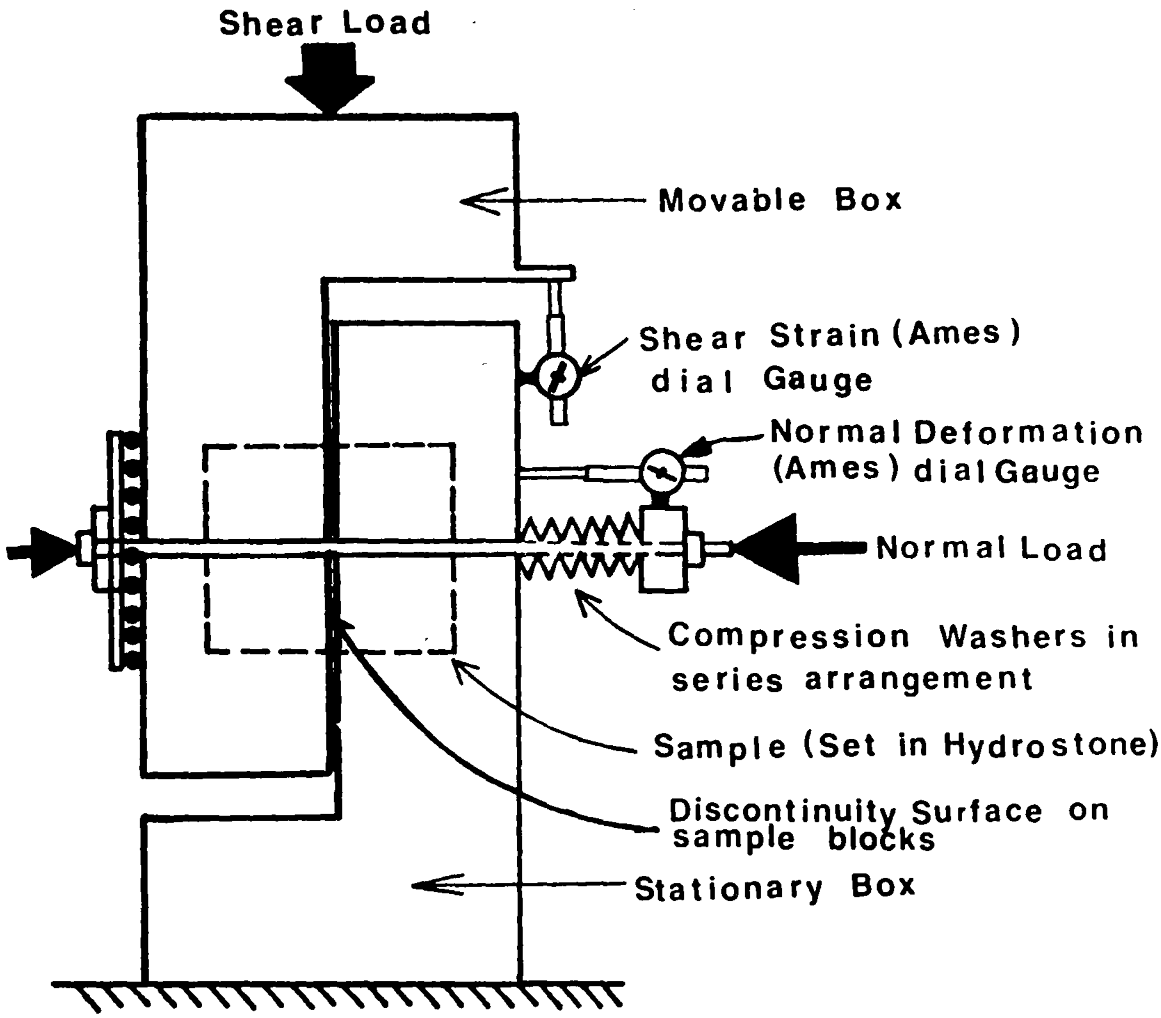
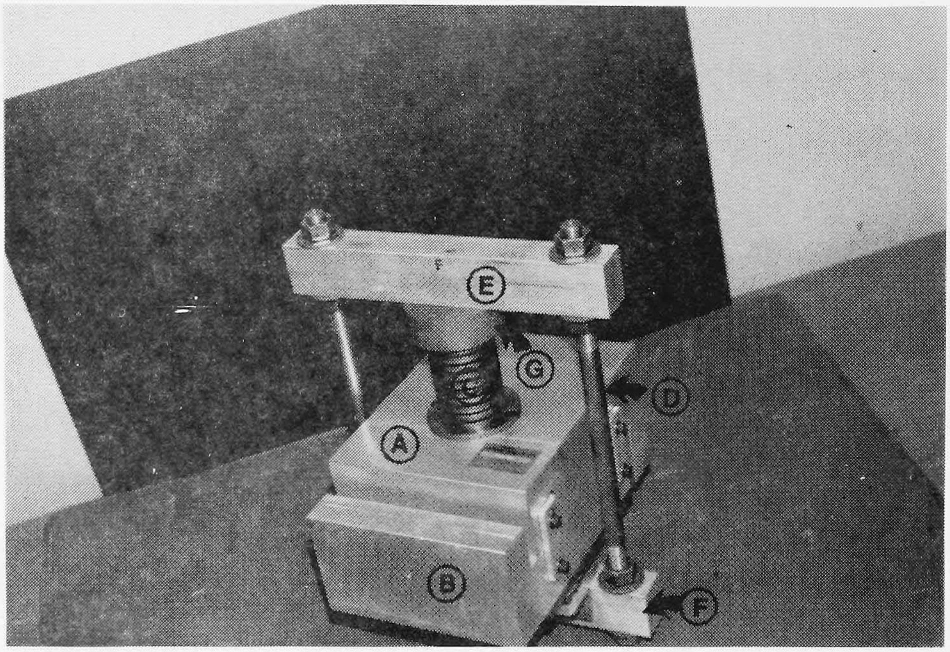


Figure 4 Diagrammatic representation of the arrangement for the direct shear test.



A = Stationary box
B = Movable box
C = Belleville Washers(20) in series arrangement
D = Vertical fastening rod
E = Upper loading platform
F = Lower loading platform
G = Hardened steel guide

Figure 5. Labeled photograph, direct shear apparatus for this study.

obtain the normal load. During the test, the normal displacements were measured by two dial gauges at both sides of the shear box. This was necessary inorder to account for rotational movements during the test. The shear force was applied and measured with a Baldwin-Tate-Emery (BTE) Universal Testing machine whereas the shear displacement was measured between the fixed loading frame and the moving loading table of the Universal Testing Machine.

Before each test was begun, an estimate of the roughness of each joint surface in terms of the angle (i) was obtained.

The testing procedure followed the standard specified by the International Society for Rock Mechanics (8) and consisted of three test runs per discontinuity (i.e. staged testing); with each run having a constant normal stress that was higher than the preceding run. Shearing was conducted beyond peak shear stresses but not to a complete residual resistance.

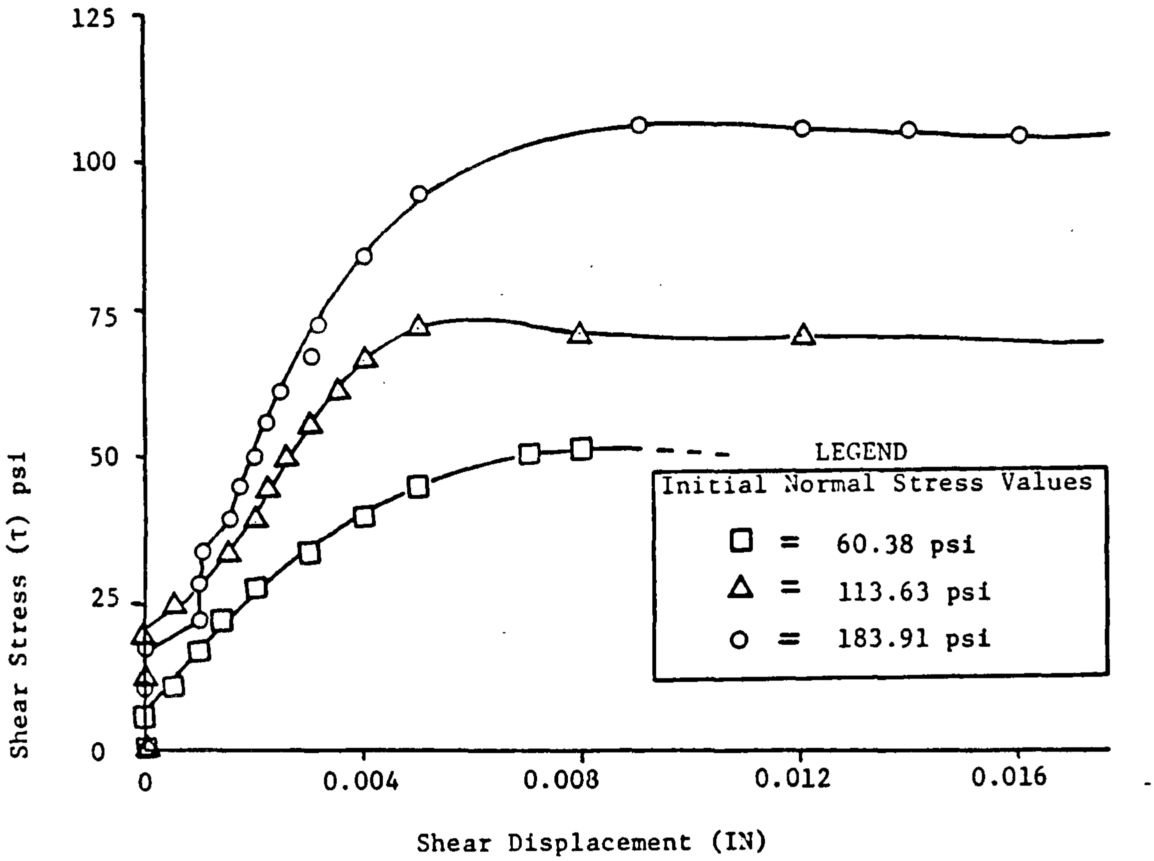
TEST RESULTS

For each test specimen, the following plots were produced: (a) shear displacement versus shear and normal stresses, (b) shear stress versus normal stress. These plots are available for seven tests but only two tests are presented graphically here in Figures 6 through 9. The results of the direct shear test are tabulated in Table 2. These results indicate that values of friction angles for the fine-grained sandstone ranged from 17.5^{o} to 36.8^{o} with a mean value of 24.4^{o} (and a standard deviation of 6.19^{o}), whereas the shear strength intercept (C_d) varied from 0 to 23.95 psi with a mean value of 8.85 psi.

DISCUSSION AND INTERPRETATION OF RESULTS

The rather wide range of values of friction angles (ϕ) and shear strength intercepts (C_d) is probably due to the fact that the joint surfaces had asperities with various degrees of roughness. Thus, the values of the tabulated friction angles in Table 2 which are higher than the mean value could be represented by the relationship $\emptyset_{eff}=(\emptyset_u+i)$ according to Patton's law (5,9) for joint shear strength, where $\emptyset_{eff}$= effective friction angle for the joint; $\emptyset_u$=friction angle of a smooth joint ($\approx\emptyset_r$ which is the residual friction angle for intact rock specimens); and i=angle of inclination of the asperity relative to the mean joint plane (see Fig. 10).

The values of the angle of inclination of asperities for the joint surfaces of the sandstone tested were estimated at about $0\text{-}3^{o}$. Hence it



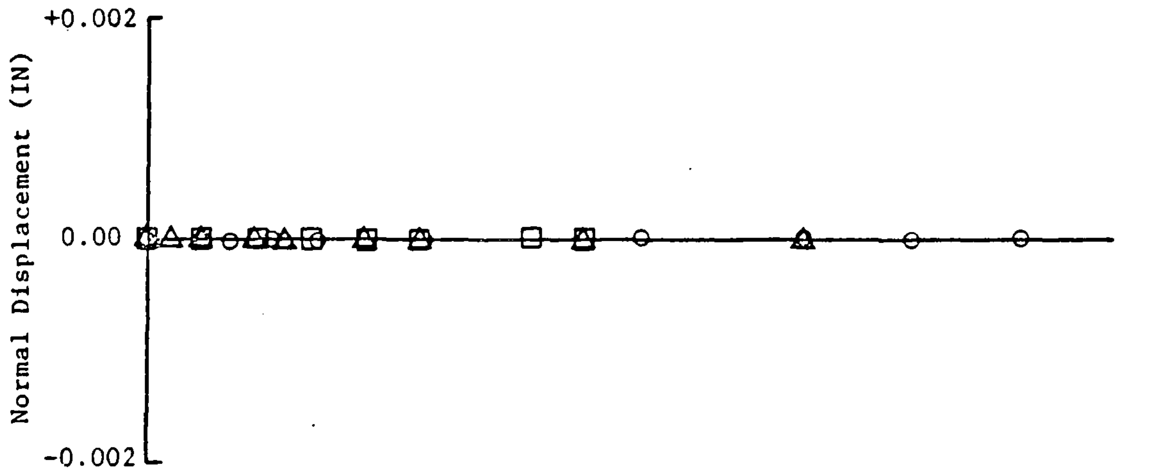


Figure 6. Direct shear test on fine-grained sandstone. Shear stress ( ) and normal displacement versus shear displacement. Test #1.

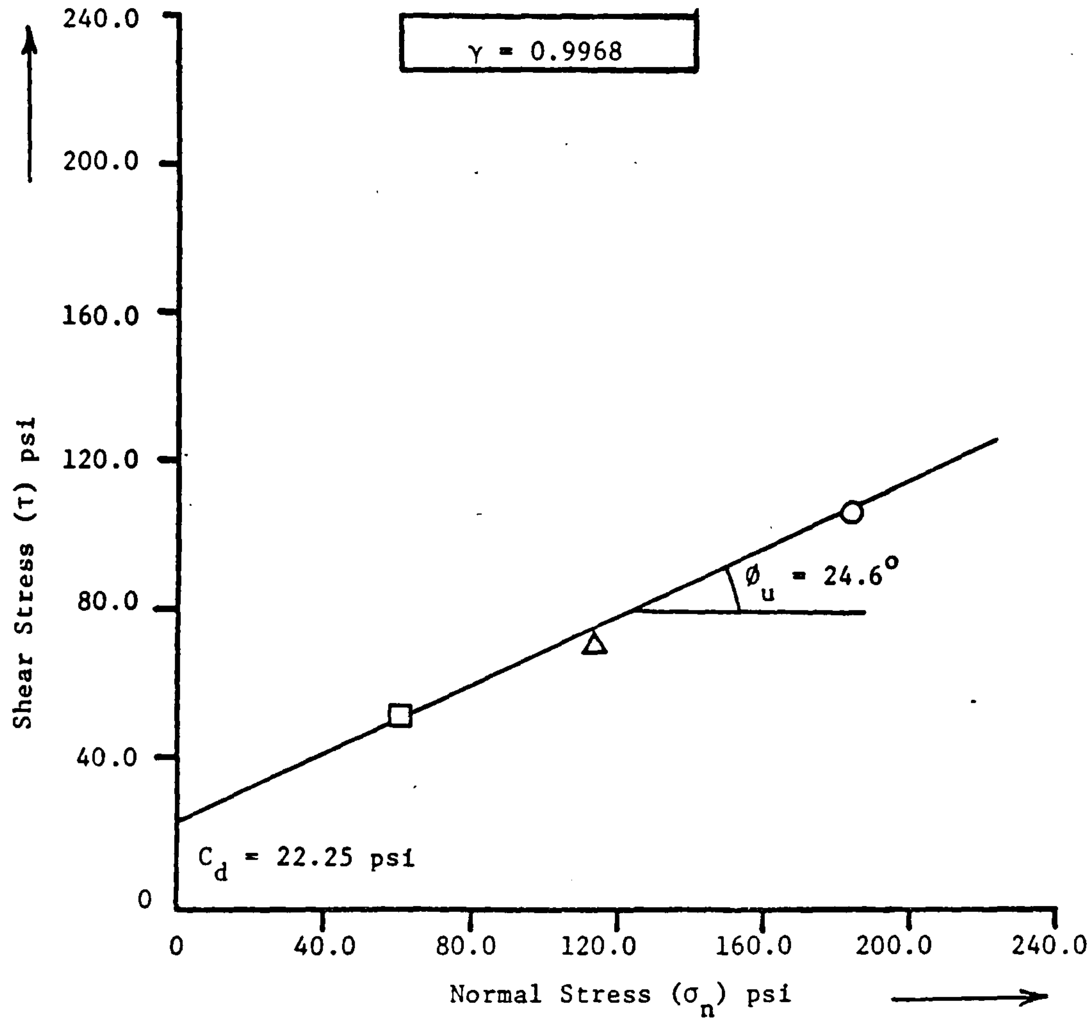


Figure 7. Direct shear test on a fine-grained sandstone from Narrow-Lake, Greene-Sullivan County, Indiana. Test #1

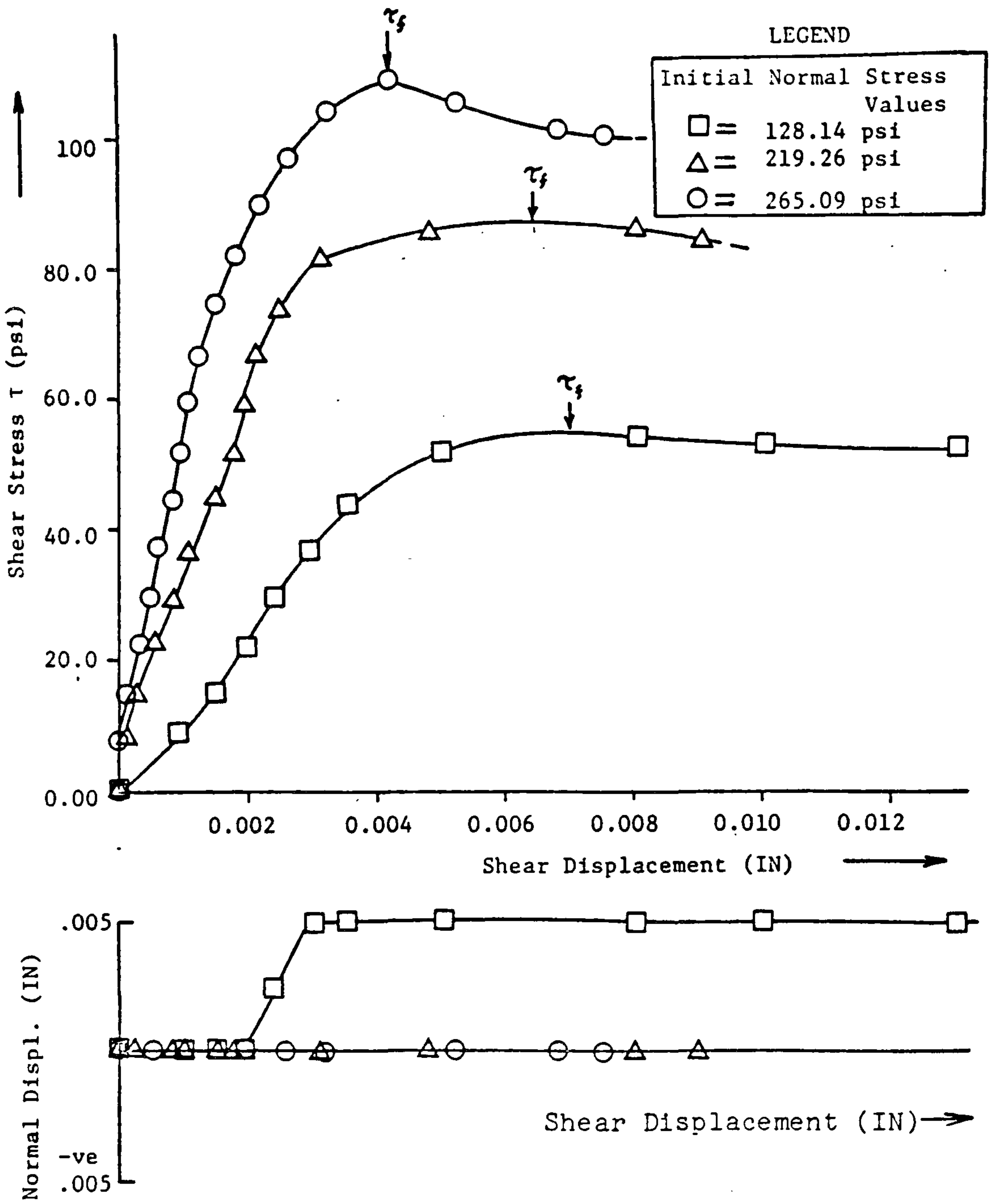


Figure 8. Direct shear test on fine-grained sandstone. Shear stress (τ) and normal displacement versus shear displacement. Test #5.

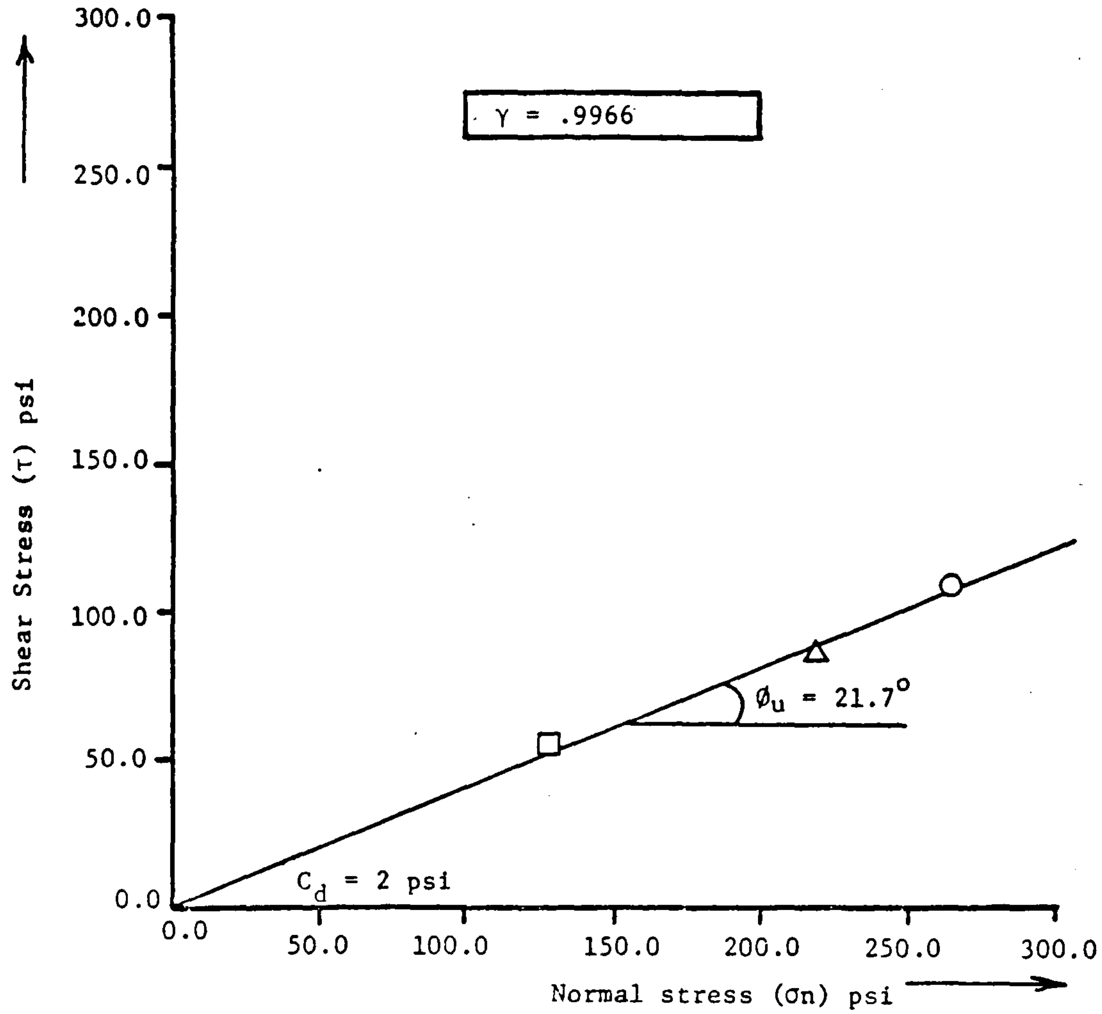


Figure 9. Direct shear test on a fine-grained sandstone from Lonnie-Lake, Greene-Sullivan County, Indiana. Test #5

Table 2. Result of direct shear test on fine-grained sandstone.

Test #	Sample #	Material Type	Range of Normal Stress (psi)	Friction angle (Ø)	Shear Strength Intercept (C_d or S_j) (psi)
1	NL1	Fine-grained sandstone	60.38-183.91	24.6^o	22.25
2	NL2	Fine-grained sandstone	53.92-152.03	36.8	7.74
3	NL3	Fine-grained sandstone	98.33-413.40	21.4^o	0.00
4	LL1	Fine-grained sandstone	96.44-283.8	17.5^o	8.00
5	LL2	Fine-grained sandstone	128.14-265.1	21.7^o	2.00
6	LL3	Fine-grained sandstone	93.23-359.86	27.0	23.95
7	LL4	Fine-grained sandstone	176.32-349.36	22.0	0.00

could be assumed for this particular case that $(\emptyset_u+i)\simeq\emptyset_r$. The range of friction angles (Ø) obtained in these tests (17.5^o to 36.8^o) for fine-grained sandstone is reasonably close to the range of values (26^o to 35^o) obtained by Patton (9) in similar tests on sandstone performed with a different apparatus, and to a range of values (21^o to 40^o) mentioned by Goodman (5). However, rock type alone is not sufficient to estimate frictional resistance since different varieties of discontinuities within the same rock type may show differing resistances.

The shear stress-deformation behavior of the discontinuities tested in this study represent behaviors typical of clean but rough fractures (4).

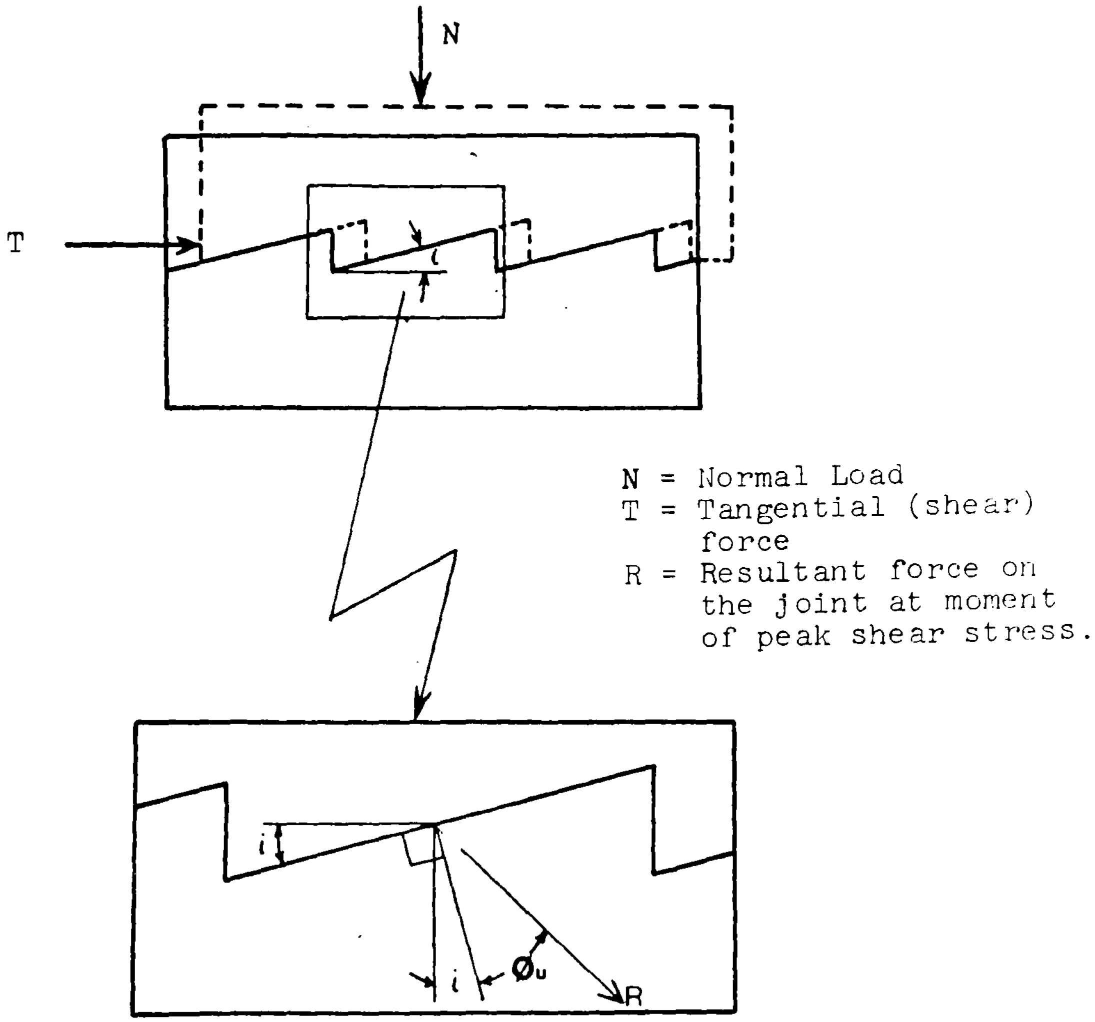


Figure 10. The basis for Patton's Law for joint shear strength (modified from Goodman, 1980).

SIGNIFICANCE OF THE VALUES OF $\emptyset$ and C_d

Shown in Figure 11 is a two-dimensional schematic representation of a highway cut with a through-going discontinuity surface BC, in a dry condition (i.e. absence of groundwater is assumed). With reference to Figure 11, the Factor-of-Safety (F.S.) of this highway cut with respect to rock slope failure is given by the relationship:

$$F.S. = \frac{C_d A}{W \sin\theta} + \frac{\tan \emptyset}{\tan \theta} \qquad (1)$$

(cf: reference 6)

where the various parameters are as defined in Figure 11.

From equation (1) it is apparent that both the shear strength parameters $\emptyset$ and C_d are important factors in the evaluation of the stability of rock slopes in highway cuts and open pit mines. High values of C_d and $\emptyset$ are required to maintain high factors of safety which are indicators of stability with respect to rock slope failures. On the other hand, low values of $\emptyset$ and C_d render rock slopes susceptible to failure.

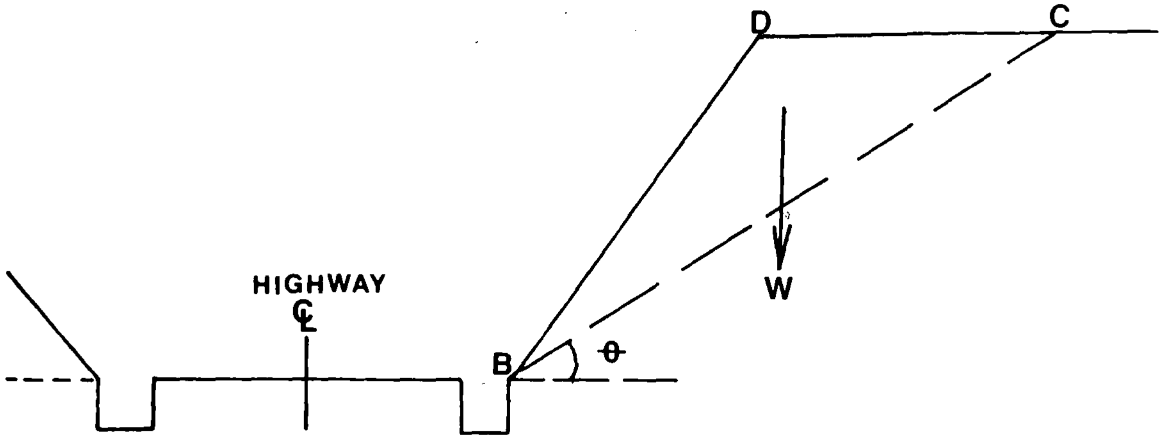
Commonly for slope stability calculations, the shear strength intercept (C_d) is not included (or assumed equal to zero) in which case equation (1) reduces to:

$$F.S. = \frac{\tan \emptyset}{\tan \theta} \qquad (2)$$

Thus, it could be observed that the friction angle of discontinuity planes ($\emptyset$) in any given rock mass governs the stability of slopes in such rock masses.

CHOICE OF FRICTION VALUES FOR DESIGN PURPOSES

The selection of a value for the friction angle for design of a particular rock slope depends, in part, on the intended life expectancy of that



BC = through-going discontinuity surface with shear strength properties C_d and ϕ

C_d = discontinuity cohesion intercept (not a physical property)

ϕ = discontinuity friction angle

θ = dip angle of discontinuity surface

W = Weight of block BCD

Figure 11 A schematic two-dimensional representation of a highway cut.

slope. For a long-term rock slope in a typical civil project, a peak value of friction angle is needed inorder to attain a conservative slope design (i.e. a high factor of safety). On the other hand, for a temporal open pit mine, a mean or representative value of friction angle is typically used for design purposes since failure of such slopes after completion of mining will not be of great consequence to either personnel or equipment.

In cases where a probabilistic analysis or one in which sensitivity of factor of safety to friction angle is required, Lilly (12) has suggested a realistic range of values (whole range of values preferred) for design purposes. This, according to Lilly, provides for the diverse friction values on the numerous discontinuity planes that occur in rock slopes. However, the probabilistic approach to the solution of rock slope problems has found little acceptance in routine slope designs probably because of the uncertainties and difficulties involved in such analytic techniques.

ADVANTAGES AND LIMITATIONS OF THE TESTING TECHNIQUE

Advantages of the testing technique used in this study include:

(a) The ability to perform stage testing (i.e. the possibility of using the same discontinuity and performing shear tests at several normal stress levels). This process is ideal for the determination of residual shear strength ($\emptyset_r$) of discontinuity surfaces.

(b) The equipment employed in this study can be used, with modifications, to study the effect of pore water pressure (or cleft water pressure) on the shear strength properties of discontinuities.

However, like most other shear strength determination techniques, there are certain limitations of this testing technique that are worthy of mention. These are:

(a) Non-reproducibility of shear strength parameters ($\emptyset$ and C_d). Once the asperities on any given joint are sheared off, the original geometry responsible for peak resistance, is lost.

(b) Stage testing ignores the effect of sheared asperity interferences. These interferences may appear in the form of gouge particles or a gouge layer, both of which would likely increase and decrease the shear resistance respectively.

(c) Problems with eccentricity may arise when small specimens are sheared in a very large apparatus.

(d) There is a finite distance (total displacement) that a test specimen can undergo during a routine test which may be considerably less than the distance involved in a slope failure. This is a common problem with most direct shear devices.

(e) Laboratory-scale discontinuity asperities may not accurately represent large scale undulations which largely control field behavior.

CONCLUSION

Results of direct shear tests on a fairly indurated, fine-grained sandstone from southwestern Indiana, have indicated that:

1. Both shear strength parameters ($\emptyset$ and C_d) and deformability can be conveniently measured with this direct shear device as the applied forces and the measured strains are related to the two-dimensional characteristics of the discontinuities.

2. The stage testing employed in this study is preferable to shearing different discontinuities at differing normal stresses since sampling of different discontinuities is both difficult and expensive to accomplish.

3. The equipment described and used in this study was found to be suitable for routine determination of shear strength parameters.

4. In addition to discontinuity shear strength determinations using small-scale laboratory samples, the persistence, attitude and spacing of discontinuities in pertinent field situations should be recorded and used in the design of highway cuts.

ACKNOWLEDGEMENTS

The testing program reported here was accomplished in the Structural Mechanics Laboratory, Civil Engineering, Purdue University. The authors wish to acknowledge the financial support provided by the Office of Surface Mining, through the Indiana Mining & Minerals Resources Research Institute. The authors also gratefully acknowledge the assistance of Mr. William Cook, technician in Civil Engineering, and that of Mr. Howard Hume plus several other graduate students in the Engineering Geology division Purdue University, during the testing program.

CITED REFERENCES

1. Barton, N.R. et al. (1978), Suggested methods for the quantitative description of discontinuities in rock masses, for ISRM Commission on Standardization of Laboratory and Field Tests, Int. J. Rock Mech. Min. Sci., vol. 15, No.6, pp. 319-368.

2. Barton, N.R. and Choubey, V. (1977), The shear strength of rock joints in theory and practice. Rock Mechanics, vol. 10, Nos.1,2, pp. 1-54.

3. Einstein, H.H. and Dowding, C.H. (1981), Shear resistance and deformability of rock discontinuities. In: Physical Properties of Rocks and Minerals, edited by Touloukian, Y.S.; Judd, W.R., and Roy, R.F.

4. Goodman, R.E. (1970), The deformability of joints. Amer. Soc. for Testing and Materials Rept. ASTM-STP 477, pp. 174-196.

5. Goodman, R.E. (1980), Introduction to rock mechanics. John Wiley & Sons, New York, 478 pages.

6. Hoek, E. and Bray, J.W. (1977), Rock Slope Engineering, 2nd edition, Institute of Mining and Metallurgy, London 402 pages.

7. International Society for Rock Mechanics (1975), Terminology. Soc. Gen. Lab. Nacional de Engenaria Civil, Lisboa, Portugal.

8. International Society for Rock Mechanics (1974). Suggested method for laboratory determination of direct shear strength. Commission on Standardization of Laboratory and Field Tests, Final Draft; February.

9. Patton, F.D. (1966), Multiple modes of shear failure in rock and related materials. Ph.D. Thesis, University of Illinois, Urbana.

10. Wier, C.E. (1952), Geology and mineral deposits of the Jasonville Quadrangle, Indiana: Indiana Geol. Survey Bull. 6, 34 pages.

11. Wier, C.E. and Gray, H.H. (1961), Geologic map of the Indianapolis $1^{\circ}\times 2^{\circ}$ Quadrangle, Indiana and Illinois, showing bedrock and unconsolidated deposits, Regional Geol. Map, Indiana Geol. Survey, Bloomington, Ind.

12. Lilly, P.A. (1982), The shear behavior of bedding planes in Mt.McRae shale with Implications for rock slope design. Int. J. Rock Mech. Min. Sci. & Geomech. Abstr., vol. 19, pp. 205-209.

Appendix A1

Calibration procedure for the

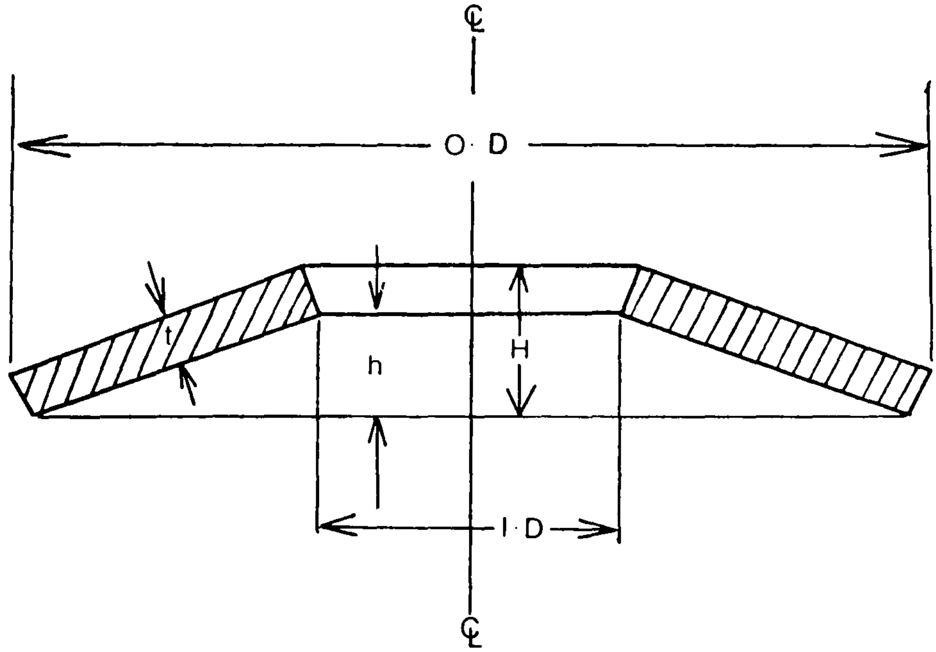
Belleville Spring Washers

Appendix A1

Calibration procedure for the Belleville Spring Washers

Introduction

The configuration of each Belleville spring washer, manufactured by the Associated Spring Corporation, is as shown in Figure A1.1. The type used is Catalog Number B2500-120, made of spring steel material and has dimensions shown in Figure A1.1.



Outside Diameter (O.D) = 2.50 in (63.50 mm); h = (H-t) = 0.06 in (1.524 mm)
Inside Diameter (I.D) = 1.250 in (31.75 mm)
Thickness (t) = 0.120 in (3.048 mm)
Load at flat position = 3200 lb (1451.5 Kg)

Figure A1.1 Configuration of the Belleville Spring Washer

Procedure

The two halves of the direct shear box with no samples inside, were assembled such that the stationary box (Figure 3.13) lay on top. A stack of 20 spring washers, arranged in series (see Figure A1.2) was installed on a rod attached to the top of the stationary box. A hardened guide was therefore placed on top of the springs. The shear box and spring assembly then was placed between the upper and lower loading platforms of a Baldwin-Tate-Emery Universal Testing machine which provided the load for the spring calibration process.

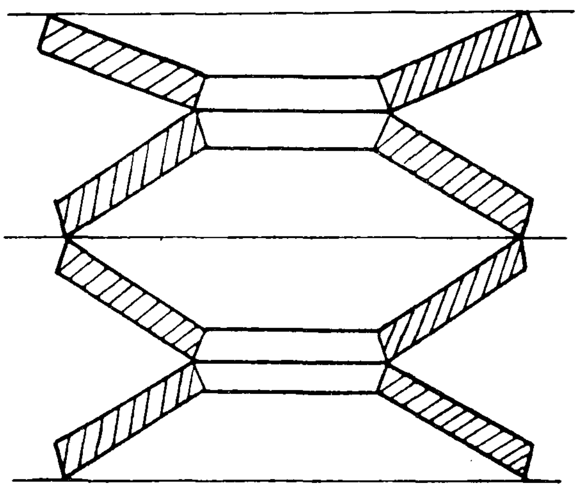


Figure A1.2 Arrangement of Belleville spring washers in series.

Load Load was applied at an average rate of about 40 Ib per minute until a dial gauge deflection of about ¼ inch was attained before unloading at an approximate rate of 40 Ib per minute also. A plot of load versus dial gauge reading was made (Figure A1.3) and the spring constant determined as 5,214.29 Ib per inch. The shear box spring system calibration data are presented in Table A1.1.

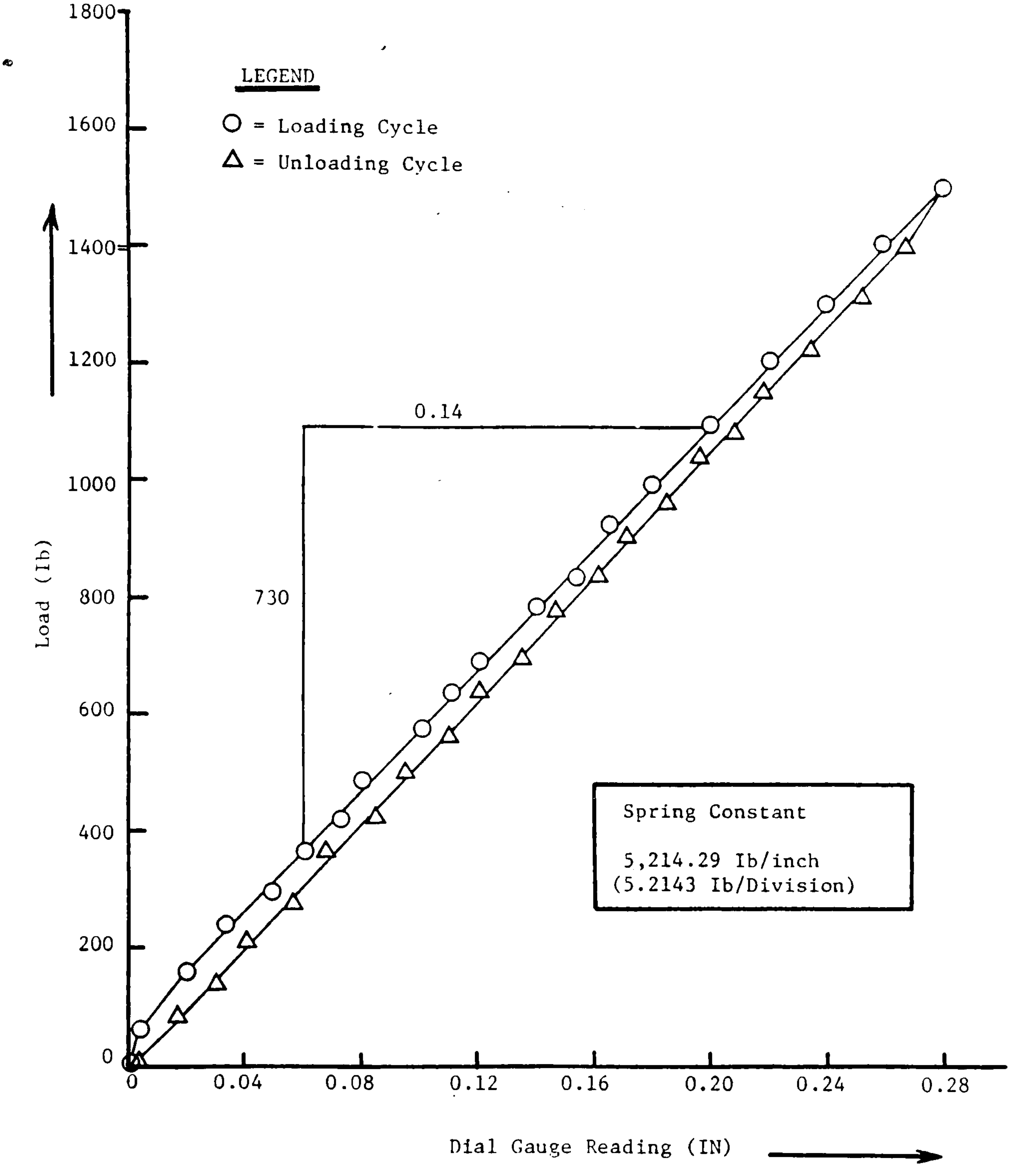


Figure A1.3 Shear-box spring calibration curve (20-washers in series arrangement).

Table A1.1 Direct shear-box spring system calibration data

**LOAD CYCLE				UNLOAD CYCLE			
Load (Ib)	Guage (x0.001")	Right Guage (x0.001")	Ave Reading (x0.001)	Load (Ib)	Left Guage (x0.001)	Right Guage (x0.001")	Ave Reading (x0.001")
0.0	0.0	0.0	0.0	1500.0	282.0	278.0	280.0
60.0	4.6	4.2	4.4	1400.0	276.0	260.0	268.0
160.0	22.05	18.0	20.0	1316.0	254.0	250.0	252.0
240.0	38.0	30.0	34.0	1220.0	239.0	233.0	236.0
290.0	51.5	48.5	50.0	1150.0	224.0	214.0	219.0
360.0	62.0	58.0	60.0	1080.0	216.2	202.2	209.2
420.0	74.05	72.0	73.0	1035.0	199.0	195.0	197.0
480.0	83.0	77.0	80.0	960.0	190.6	180.6	185.6
575.0	105.0	95.0	100.0	900.0	174.0	170.0	172.0
632.5	116.0	104.0	110.0	830.0	164.0	160.0	162.0
690.0	125.0	115.0	120.0	775.0	149.0	145.0	147.0
780.0	143.0	137.0	140.0	690.0	139.0	133.0	136.0
830.0	157.6	150.4	154.0	640.0	125.0	115.0	120.0
920.0	172.5	160.5	166.5	560.0	115.0	105.0	110.0
990.0	182.0	178.0	180.0	490.0	96.0	92.0	94.0
1090.0	202.0	198.0	200.0	420.0	88.0	80.0	84.0
1200.0	225.0	215.0	220.0	360.0	72.0	64.0	68.0
1300.0	247.0	233.0	240.0	270.0	60.0	52.0	56.0
1400.0	264.0	256.0	260.0	200.0	43.0	37.0	40.0
1500.0	282.0	278.0	280.0	140.0	34.0	26.0	30.0
				80.0	20.0	14.0	17.0
				0.0	4.0	0.0	4.0

** Loading Machine Model: Baldwin-Tate-Emery
120,000 Ib type

Average Load increment rate: 50 Ib/minute

Dial Guages used: Soil Test Ames Dial guages

COMPUTER GENERATED MAPS

A. Keith Turner

Department of Geology
Colorado School of Mines
Golden, Colorado 80401

Abstract — Geological applications of digital cartography are developing more slowly than for many other cartographic applications. A review of the unique characteristics of geological maps and existing computer hardware and software reveals that existing techniques are only marginally cost-effective in supporting traditional map production.

Conversely, the current and anticipated digital mapping technologies can be used effectively to support and enhance many engineering geology investigations. The most cost-effective methods vary with the type and stage of the engineering geological investigation, but it appears certain that a variety of developments will grow in the next five to ten years.

Introduction

Maps are fundamental to geology; from the earliest days a geological map was an important product, sometimes the only product, of a geological investigation. Map production is slow and expensive. A review of historical records of geological organizations reveals frequent complaints about the lack of skilled draftsmen, the delays in getting lithographic plates produced, and the costs.

It has been at least 20 years since computer graphics techniques were first applied to cartography. The introduction of digital techniques to cartographic processes offered the potential for solving the production problems of traditional methods. Great strides have been made, yet digital cartography has not yet replaced, to any significant extent, the traditional methods for producing geological maps.

A geological map is a very specialized document. Its formulation has been developed to suit the specific needs of a geologist. Most regional geological maps at 1:250000 scale or larger are printed on topographic base maps. Traditionally, rock units are differentiated by colors and symbols, although cost saving efforts have led to the increasing production of uncolored geologic maps. A number of specialized symbols have been developed to indicate the interrelationships among the geologic units. Geologic mapping is quite different from other thematic mapping such as land cover maps, soils maps, forestry maps, etc. Geologic mapping attempts to portray on the two-dimensional paper the three dimensional interrelationships among geologic units. The symbology to do this is necessarily complex. It is also standardized and largely internationally accepted, although some differences occur among the various national surveys. The symbology has evolved very slowly over the last century, so that maps of 50 or even 100 years ago are readily readable by the modern geologist.

The traditional geological mapping symbols have posed a real challenge to the use of computer graphics. Standard computer equipment of nominal cost either cannot reproduce these symbols at all, or can do so only in an unconvincing manner. Digitally produced geologic maps have been produced, and some are superior in cartographic quality to traditional maps, but only with the use of comparatively expensive hardware.

Digital Cartographic Procedures

System Components

Digital cartographic systems must have four components:

1) Data entry - a method of converting graphical products into a numerical format suitable for entry into a computer, usually through a process called "digitizing".
2) Data storage and retrieval - the numerical cartographic data constitutes a data base, whose organization must be carefully formulated to allow the user to efficiently extract desired combinations of information.
3) Data editing - the user must be able to examine the data base and to make changes and corrections, preferably in an interactive fashion.
4) Data display - the user must be able to produce suitable map products having the information content, scale, and map projection as desired.

Manual verses Digital Procedures

There is a fundamental difference between manual and digital cartography. With manual techniques the map is the primary product; while with digital techniques the numerical data base is the primary product, and maps are secondary products. Digital systems are rarely if ever cost-effective if only a single map product is to be produced; rather they become attractive because of their selective editing capabilities. Maps with a variety of information contents, or scales, or projections can be produced from a single data base. The data base can be selectively updated and new products produced.

The above comments suggest both the advantages and disadvantages of digital cartography. Digital cartography is cost effective where multiple products must be produced, or where the user requests are variable, or where the map data are changing with time (such as land cover data), or where the map is but one product and the data base can be used for other purposes, such as physical modeling for instance. Conversely, manual techniques are usually cheaper where a single map is to be produced and it will remain unchanged for a long period of time, or if scale changes would invalidate the suitability of the data being mapped.

Of course cost is not always the only concern, although it is generally an important one. Digital systems, once fully operational, give the users the option of speeding up the production process. Most digital drafting devices can produce a drawing much more rapidly than a human, and they can work multiple shifts, and do not take coffee breaks. Digital systems allow the users the options of producing different products, ones which cannot be produced by manual techniques. This additional capability is hard to quantify but is a real benefit in many situations.

Digital Data Base Design

Of the four components in a digital cartographic system, the data base design is probably the most critical to the overall success of the system. A poorly designed data base will just not be able to function and produce the products the user desires. Data entry, edit, and display functions can be changed or upgraded over time as user demands change, but the data base structure represents a large and growing investment and it must be suitable.

One problem with cartographic data base design is the large size of these data bases. Rhind and Adams (1981) give a very instructive review of the problem. They quote Doyle (1978) who pointed out that a gridded elevation model of the Earth's land surface with one point per meter would require 1.6 million magnetic tapes to hold the data and take three years of computer time to read! While such a data base is clearly unworkable, we are producing vast amounts of remote sensor data by LANDSAT for example. The World Data Base II, a numerical outline map of the world's coastlines and related data developed for the CIA, has over 5 1/2 million points located by latitude-longitude. The organization of such data bases is not a trivial task.

Three Competing Data Base Formats

Almost from the beginning of digital cartography there has been debate concerning the most appropriate format for cartographic data bases. Three approaches have been championed, derided, resurrected, and have endured. They are:

1) Gridded, or cellular, or raster data
2) Vector, or arc, or polygonal data
3) Triangular mesh data

The detailed discussions of the merits of these quite different approaches are beyond the limits of this paper. It should be pointed out, however, that much of this discussion revolves around the use of these techniques for topographic mapping.

The gridded data method is a popular technique, especially for contouring. It has a number of computational advantages, at a cost of increased data storage. This last concern is diminishing as the cost of memory hardware drops. Gridded or raster data are desired by many of the newer display devices, including refresh screen CRT's, electrostatic plotters, and ink-jet plotters. As the resolution of these devices improves and competes with the more traditional pen plotter devices, raster data structures appear more desirable.

For many years, the best quality computer graphics display devices were vector oriented units - pen plotters of the drum or flatbed variety, and storage tubes. These devices coincided with the still expensive computer memory restrictions of the early 1970's to boost the interest in vector data. The procedure was analogous to the classic map making procedures.

Triangular mesh techniques have always been the "third-force". The early IBM contouring programs used this method, and it continued to be the subject of research throughout the early 1970's. Several commercial applications are based on this method (Males and Gates, 1980), including one for handling geological subsurface data (Gold, 1980).

Review of Pertinent Developments in Digital Cartography

Geologists are interested, to varying degrees, in a variety of digital cartography developments. While geological maps are definitely thematic maps, the geologist is often dependent on topographic maps to form his map base. Frequently a map base at suitable scale is unavailable, in which case photographic enlargement or reduction of one or several maps is undertaken to provide such a base. Such procedures often violate map accuracy standards, yet the geologist rarely concerns himself with such matters, perhaps because the positional reliability of many of his geologic contacts is much less than any topographic features. Geologists also extensively use aerial photographs and remote sensing imagery.

Digital Topographic Mapping

By the early 1970's a number of digital topographic mapping systems were in place and operational at civil engineering organizations. One of the earliest was at the Ontario Ministry of Transportation and Communications which became operational in 1970 (Turner, 1981). Other systems have been installed in such states as California, Texas, Georgia and Michigan. All of these sytems were concerned with the production of large scale (1:500 to 1:2500) construction plans. A logical extension of these technologies has led to the production of utilities maps by utility firms or municipalities, such as Houston. Boyle (1981, p. 32) points out that there is a difference between such mapping, which closely emulates engineering drawings and other CAD/CAM applications, and national topographic mapping programs. Boyle suggests that the size and complexity of the spatial relationships among map features is so much greater for standard map quadrangles that the earlier digital map drafting procedures will not be satisfactory.

Some experimentation by national mapping concerns, chiefly in Canada (Zarzycki, 1978) and Britain (Rhind and Adams, 1981) led to the implementation of a number of experimental installations. The Canadians have installed a system designed to map at 1:50000 scale in the Arctic and produced maps beginning in 1978 (Zarzycki, 1978).

These systems utilized high resolution flatbed plotters to develop the map plates for printing. Such maps meet or exceed the drafting quality of traditional methods. The development of such systems became increasingly affected by the development of analytical plotters which allowed essentially automated map production. These were introduced in the 1960's but they were, and have remained, extremely expensive, so that by the late 1970's only a few were operational. Their presence is profoundly affecting the longer range planning of national mapping agencies (Zarzycki, 1978; Case, 1981; Southard, 1980; Williams, 1980).

Digital Thematic Mapping

Thematic mapping covers an enormous variety of applications and there are a variety of sytems designed to meet some portion of them. Dudycha (1981) gives a most readable overview of these various types of systems, and the reader is referred to the various volumes of the Harvard Library of Computer Graphics for many examples.

On the one extreme are low cost, low resolution cellular systems producing graphical products on the line printer; while on the other extreme are sytems designed to produce full color maps rivaling the best a human draftsman can produce.

Many thematic mapping systems involve the combining or compsiting of several map products to produce new themes. Polygon intersection algorithms do exist, but most commercially available ones tend to operate inefficiently when very large numbers of polygons are encountered. Raster, or cellular, operations are often more efficient but this requires vector to raster and raster to vector conversions which may be expensive (Peuquet, 1981a; 1981b).

Tomlinson and Boyle (1981) describe a review of most major commercial thematic mapping systems undertaken for the Saskatchewan government. This review concluded that none of the existing systems met the desired operational criteria and recommended that no system be purchased by Saskatchewan at this time.

In spite of the undoubted problems documented by Tomlinson and Boyle, research into thematic mapping continues. The Experimental Cartography Unit (ECU), established in London England in 1967, has developed a number of thematic maps; including several British geologic maps (such as the Abingdon map produced in 1970) and a folio of maps jointly with the Kansas Geological Survey (Campbell, et al; 1979).

In the United States much work has been done by the U.S. Forest Service, the U.S. Fish and Wildlife Service, the U.S. Geological Survey for land use and land cover mapping (Guptill; 1981), and the Soil Conservation Service (Johnson; 1980). All these products use vector oriented data bases, with the exception of some of the Forest Service products. Digital cartography has been used primarily to speed up the production of the map products. The maps are still looked on as a primary product and the digital data base appears secondary. While these data bases are publically available, it is not clear to the author at this time whether they are being used to any extent by second or third party users. The lack of software, standardized data formats, and the incompleteness of coverage to date have tended to limit the utility of these data bases.

Software and Hardware

It is not the intent of this paper to review software or hardware availability. Calkins and Marble (1980) have recently issued a three-volume index of software; while Petrie (1981) has reviewed many important hardware trends.

Obviously the availibility of suitable software and hardware is a necessary condition for the successful use of digital cartography. Most geologists and geological agencies have neither the expertise nor the desire to invent such systems.

A number of trends in technology development promise to encourage the greater use of digital cartography in the near future. We have an enormous amount of existing traditional map data. How can such data be rapidly digitized? Manual line following digitizing appears much too inefficient. Fortunately, a number of scanning systems are now available which can accomplish this task efficiently (Montuori, 1980; Leberl and Olson, 1982).

Optical disk technology is now advanced to the point where home video playback systems are on the market. Such systems fare poorly on the home video market because recording requires very expensive equipment and "clean room" conditions. However, they have the ability of storing vast amounts of map data very economically, without discernable deterioration over time, and capable of very rapid access to any desired scene. These are predicted to be the master storage medium for much topographic data (Southard, 1980).

Assessment

Digital Cartography for Traditional Geological Maps

At present, the use of digital cartography to produce traditional geologic maps is very limited. Greater use of appropriate technology by the Soil Conservation Service and the Geography Program of the U.S. Geological Survey demonstrate the technical feasibility, as does the Kansas Geological Survey experimental quadrangle (Campbell et al, 1979). Why then are the geological organizations not using this technology more? The answer appears to lie partly in the conservatism of many geologists and geological organizations, and partly in the economics.

Geology maps represent the results of lengthy field investigations and their production may take years. Users are not as readily apparent for clamoring for the data, as are the agricultural interests for the soils surveys. Geology maps do not change with age and the survey is usually done with a precision commensurate with the expected map scale, so the ability to enlarge or reduce the map scale is minimal value.

These factors appear to minimize the desire for geological organizations to invest in the very expensive hardware needed to produce traditional geologic map equivalents using digital methods.

Digital Cartography for Engineering Geology

Engineering geology mapping requirements frequently differ from those associated with traditional geological mapping. No universal mapping symbology has developed for engineering geology mapping, although the needs and techniques developed to date have been, and continue to be, the subject of much debate within the profession (Varnes, 1974; Keaton, 1982).

Many of these mapping techniques can be enhanced by the concurrent development of geotechnical data banks to facilitate computer searches and data manipulations (Goldberg et al, 1980; Lo and Lovell, 1982).

The engineering geology mapping requirements differ greatly among different types of projects and among the various design stages for each project. The mapping needs for a large dam differ from those of a new community, while the early stages of all projects require mo[illegible] of larger areas than for the later final design. These differences are reflected in the choice of the most cost effective digital mapping system for each project and planning stage.

In the early stages of project feasibility planning (often called "preliminary planning", "site selection", "pre-engineering" or "preliminary design") there is a need to evaluate and compare comparatively large areas. Often geological and non-geological factors must be compared for several sites as part of an environmental assessment. Such assessments may be mandated by governmental regulations, or required by the policies of the client or the lending agencies. The latter case is especially true when projects are being funded by one of the international banks supporting developments in third world countries.

The needs of such studies are often best served by comparatively simple cellular mapping and compositing systems (Schneider and Amanulla, 1979; Turner, 1976). The cellular mapping systems are economical to use and do not require expensive specialized hardware. Furthermore, they can be combined with optimization procedures to perform site selection criteria rankings (Turner, et al, 1981) or corridor selection for highways, pipelines or transmission lines (Turner, 1978). These approaches lend themselves to the development and testing of conceptual alternatives, to the production of environmental assessment reports, and to the presentation of recommendations to public forums and decision-makers with a wide variety of backgrounds.

In the later stages of project design, engineering geologists are usually involved in detailed analyses of much smaller areas. The required precision of the graphical displays points toward different digital cartographic approaches. Methods of handling, cross-indexing, and displaying a variety of sample data, including those from boreholes, generally involves much more sophisticated programming software. The display of maps, cross-sections and isometric views, while important to the engineering geologist as an aid in making and presenting his interpretations, generally will require specialized and expensive display equipment. Such applications are still more experimental than a production reality but progress is occurring in this area and production systems can be expected shortly. The impact of such computer capabilities on proposed three dimensional mapping methods is likely to be great (Kempton, 1981; Kempton and Berg, 1982).

The Future for Digital Cartography in Geology

I anticipate an expansion of the use of digital cartography methods by geologists. More and more geological data are being processed by computer as a result of routine data management procedures. Results for a series of rock samples are stored in a computer as part of a "word processing" sequence, or for convenience in transmitting data from laboratory, to office, to field. Once entered, the data need only the addition of coordinates to be plotted. Geologists still have a graphical, or cartographical, bias and the urge to see the results in correct spatial relationships is overwhelming.

The production of "traditional" maps may be more slow in coming, but I expect to see increased digital activity in this area also. The availability of digital terrain map products is going to increase quickly in the developed countries and once it is available, the need to merge geological and other thematic data with the topographic data will become obviously desirable. The availability of methods of rapidly digitizing existing older maps, via scanner techniques, and the ability to create large browse capabilities using optical disk technology with digital data base creation on demand, will encourage the trend toward digital techniques.

Such developments are likely to be constrained in the near term by a lack of financial resources to purchase the expensive hardware required, and by the lack of suitable software (Petrie, 1981; Zarzycki, 1979). There appears to be less of a restriction concerning available cartographic data bases (Burkart, 1981; Rhind and Adams, 1981).

References Cited

Bickmore, D.P., 1975; The relevence of cartography: Display and Analysis of Spatial Data (J.D. Davis and M.J. McCullough, eds.), J. Wiley and Sons, p. 328-351.

Boyle, A.R., 1981, Concerns about the present applications of computer-assisted cartography: Cartographica vol. 18, no. 1, p.31-33.

Burkhart, B.R., 1981; Cartographic base files at Lawrence Berkeley Laboratory Harvard Library of Computer Graphics, vol. 13, p. 11-46.

Calkins, H. and Marble, D.F., 1980; Computer software for spatial data handling: Intl. Geographical Union, Commission on Geographical Data Sensing and Processing, Ottawa Canada (3 vols.).

Case, J.F., 1981; Automation in Photogrammetry: Photogrammetric Engineering and Remote Sensing, vol. XLVII, no. 3, p. 355-364.

Doyle, F.J., 1978; Digital terrain models: an overview: Photogrammetric Engineering and Remote Sensing, vol. XLIV, no. 12, p. 1481-1485.

Dudycha, D.J., 1981; The impact of computer cartography: Cartographica, vol. 18, no. 2, p. 116-150.

Feineman, D.R., 1980; A case study in computer mapping in petroleum exploration : Harvard Library of Computer Graphics, vol. 10, p. 41-51.

Gold, C.M., 1980; Drill-hole data validation for subsurface stratigraphic modeling: Harvard Library Computer Graphics, vol. 10, p. 52-58.

Goldberg, G.D., Lovell, C.W. and Miles, R.D., 1980; Computerized Information System for Indiana Soils: Transportation Research Record 733, pp. 74-82

Guptill, S.C., 1981; Thematic map production from digital spatial data: Harvard Library of Computer Graphics, vol. 15, p. 121-124.

Johnson, C.G., 1980; National Soils data base from the Soil Conservation Service: Harvard Library of Computer Graphics, vol. 8, p. 21-30.

Keaton, J.R., 1982; Genesis-Lithology-Qualifer (GLQ) System of Engineering Geology Mapping Symbols: Transportation Research Record (in press).

Kempton, J.P., 1981; Three-dimensional Geologic Mapping for Environmental Studies in Illinois: Env. Geol. Note 100, Ill. State Geol. Survey, 43 p.

Kempton, J.P. and Berg, R.C. 1982; Three-dimensional Geologic Maps -- A Basis for land use Interpretations: Illinois Map Notes No. 3, Lab. for Cartography and Spatial Analysis, Dept. Geography, Northern Illinois University, De Kalb, Ill. June 1982.

Leberl, F.W. and Olson, D., 1982; Raster scanning for operational digitizing of graphical data: Photogrammetric Engineering and Remote Sensing, vol. XLVIII, no. 4, p. 615-628.

Lo, Y-K.T. and Lovell, C.W., 1982; The Geotechnical Data Bank: Proceedings 31st Annual Highway Geology Symposium, Austin, Texas, pp. 105-120.

Males, R.M. and Gates, W.E., 1980; ADAPT: A digital terrain model-based geographic information system: Harvard Library of Computer Graphics, vol. 8, p. 182-196.

Montuori, J.S., 1980; Image Scanner Technology: Photogrammetric Engineering and Remote Sensing, vol. XLVI, no. 1, p. 49-64.

Peuquet, D.J., 1981a; An examination of techniques for reformatting digital cartographic data. Part 1: the raster to vector process: Cartographica, vol. 18, no. 1, p. 34-48.

Peuquet, D. J., 1981b; An examination of techniques for reformatting digital cartographic data. Part 2. The vector to raster process: Cartographica, vol. 18, no. 3, p. 21-33.

Petrie, G., 1981; Hardware aspects of digital mapping: Photogrammetric Engineering and Remote Sensing, vol. XLVII, no. 3, p. 307-322.

Rhind, D. and Adams, T., 1981; Coordinate data bases: availability and characteristics: Harvard Library of Computer Graphics, vol. 13, p. 53-60

Schneider, D.M. and Amanulla, S. 1979; Computer Assisted Land Resources Planning: Report 339, Planning Advisory Service, Amer. Planning Assoc., Chicago, 46 p.

Southard, R.B., 1980; The changing scene in survey and mapping: Photogrammetric Engineering and Remote Sensing, vol. XLVI, no. 11, p. 1415-1420.

Tomlinson, R.F. and Boyle, A.R., 1981; The state of development of systems for handling natural resources inventory data: Cartographica, vol. 18, no. 4, p. 65-95.

Turner, A.K., 1976; Computer Aided Environmental Impact Analyses (Parts 1 and 2): Colorado School of Mines Mineral Industries Bull., v. 19, Nos. 2 and 3.

Turner, A.K., 1978; A Decade of Experience in Computer Aided Route Selection: Photogrammetric Engineering, vol. XLIV, no. 12, p. 1561-1576.

Turner, A.K., Weber, J.C. and DeAngelis, M., 1981; A Geographic Market Suitability Analysis for Low and Intermediate Temperature Solar IPH Systems: Solar Energy Research Institute, TR733-1194, vols. I and II.

Turner, J.B., 1981; Computer aided photogrammetry at M.T.C.: Workshop for Automated Photogrammetry and Cartography; Amer. Society Photogrammetry, Falls Church, VA., p. 73-80.

Varnes, D.J., 1974; The Logic of Geological Maps, with reference to their interpretation and use for engineering purposes: U.S. Geol. Survey Prof. Paper 837, 48 p.

Williams, O.W., 1980; Outlook on future mapping, charting, and geodesy systems: Photogrammetric Engineering and Remote Sensing, vol. XLVI, no. 4, p. 487-494.

Zarzycki, J.M., 1978; An integrated digital mapping system: Canadian Surveyor, vol. 32, no. 4, p. 443-452.

Zarzycki, J.M., 1979; Opportunities and prospectives in digital mapping and automated cartography: Proceedings USGS Centennial Symposium, Reston (in press).

EVALUATION OF DIFFERENTIAL SETTLEMENT OR COLLAPSE POTENTIAL

by
Richard C. Benson, C.P.G.
TECHNOS, Inc.
Miami, Florida

INTRODUCTION

Differential settlement or collapse because of fractures, faults, piping, cavities, and changing soil conditions are problems commonly associated with road construction and maintenance. Ideally, these geotechnical problems should be identified so that they may be solved in the pre-construction stages. Unfortunately, the evaluation of post-construction problems and remedial action is commonly required because the problem was not identified initially.

Recognition, location, and evaluation of these natural features continues to be a problem because they are localized and not easily recognized in aerial photos or ground observations. Because of their size, they also are extremely difficult to detect by a drilling program.

Localized conditions, which can cause differential settlement or collapse, can be as simple as a change in soil condition or may be related to a deeper cavity system. Fractures/faults or soil piping may, in themselves, be a problem or may interact between a deeper cavity system and the surface soils.

If the world were all "layer cake" geology, our investigation problem would be minimal. However, heterogeneities in the natural soil and rock system are common. If heterogeneities are small, then the possibility of missing them in a field investigation is high. Subtle changes of only a few percent in soil types may go undetected, and yet may have an order of magnitude influence on hydraulic conductivity. These small changes are seldom detected in a normal field investigation. By missing such localized features, the accuracy of the overall investigation may be unknowingly less than that required.

Improving accuracy of subsurface investigation to acceptable levels usually will require extensive sampling. Spatial sampling requirements to "hit" an anomalous condition can be established, if we can estimate the size of the "target". For example, if a 10-acre site had a possible localized sub-surface feature (organic deposit or old sinkhole) of one acre and the objective was to detect the anomalous conditions, approximately 12-to-15 borings on a grid would be required to achieve detection probabilities approaching 100%. If drilling locations are randomly selected, rather than located on a grid pattern, the number of borings required can increase by a factor of three-to-four to achieve the same probability of detection.

A common geotechnical sampling approach for soil investigation is to use five borings, one in each corner of the site and one in the middle. In the case of a 10:1 site-to-target ratio with five borings, the probabilty of detection is about 60%. In the case of 100:1 site-to-target ratios, the detection probability would be less than 10%. For 1000:1 ratios, the probability of a hit is less than 1%. A 10:1 ratio is uncommon for many natural features, and ratios of 100:1, 1000:1, or greater are typical.

Further, variations in soil and rock profiles and character occur in both horizontal and vertical directions. These anomalous conditions can range from macroscopic to microscopic sizes, which make the problem of estimating size and location a difficult one. It is obvious that to achieve a good statistical evaluation of complex subsurface conditions, borings will need to be placed in a close-order grid, which could reduce the site to "swiss cheese". While critical projects, such as dams and nuclear plants, may justify high density drilling coverage, most investigations do not. In fact, we normally are dealing with a very low number of samples which result in interpretations based on assumptions which are not necessarily representative of site conditions and can lead to significant errors.

The detection of subsurface fractures, cavities, piping, or simply a local change in soil condition is not a simple problem. If these features are small, the problem becomes one of looking for the proverbial "needle in a haystack". There is, however, a wide range of technology and methodology to aid subsurface investigation. They may range from simple surface observations of changes in vegetation to an extensive drilling program. The best approach is one in which a broad range of skills and technology is applied to bring about an economical and technically optimum solution.

A SYSTEMS APPROACH

A new and proven systems approach has evolved which is capable of significant improvements in detailed subsurface investigations and the identification of localized problem areas. Once localized geologic features have been identified and evaluated, effective design or remedial action programs can be initiated. This integrated approach has provided:

- rapid solutions to geotechnical problems
- cost-effective site assessments
- improved confidence levels
- minimal risk
- early warning of hazardous conditions

The first step in applying the systems approach is to perform a mission analysis by reviewing the project objectives, schedule, and known information about the site. The systems approach does not have to be an extensive effort in terms of cost or time; however, it should include:

- definition of the mission and sub-tasks
- identification of alternative techniques/tools
- trade-off evaluation of alternatives
- design of optimum field program
- implementation

AVAILABLE TOOLS AND TECHNIQUES AND SKILLS

Alternative investigation techniques are available. They are summarized in Figure 1 and include:

- o use of existing data and aerial photos
- o traditional direct sampling/drilling and lab testing
- o on-site observations and local information
- o contemporary direct sampling methods and in-situ sensors
- o analytical capabilities
- o downhole geophysical methods
- o surface geophysical methods
- o airborne remote sensing
- o geology and earth sciences
- o engineering soil and rock mechanics

REMOTE SENSING -- GEOPHYSICAL METHODS

Among this wide range of techniques available, remote sensing/geophysical methods stand out as being particularly useful.

Airborne or satellite remote sensing clearly has benefits in terms of spatial coverage per unit time and cost. Subsurface data is obtained only by interpretation.

Surface geophysical methods yield less spatial coverage per unit time and cost, but can significantly improve resolution while providing subsurface information. A "three-dimensional" picture often can be generated using special measurement techniques.

Downhole methods (lowering various sensors down boreholes) will further improve resolution of local details. The volume sampled usually is much less than attainable by surface methods, although much more than that achieved by drilling alone. The major benefit of downhole methods is that very detailed information may be acquired at significant depths. The method, however, is very costly per unit area of coverage.

TWO CONTEMPORARY GEOPHYSICAL METHODS

In many cases, a balance between high-density spatial sampling requirements and a cost-effective drilling program can be achieved by utilizing surface geophysical techniques. Two contemporary geophysical tools, Ground Penetrating Radar (GPR) and Electromagnetic Conductivitity Measurements (EM), provide unique capability in that they offer a means of obtaining high resolution subsurface information. These two methods offer an effective approach for reconnaissance and detailed site investigation because of their continuous site coverage and the rapid traverse speeds available. Speeds up to five mph or more may be obtained; higher speeds are applicable for reconnaissance surveys, slower speeds for more detailed studies. Additionally, these methods provide in-situ (non-destructive) measurements. Both methods are somewhat limited in depth (less than 15 meters).

Radar is a reflection technique, which results in a graphic profile picture of subsurface conditions. It offers the highest resolution of all the surface geophysical methods -- although performance is site-specific, one-to-ten meters are reasonable penetration depths.

The EM technique permits rapid measurements of bulk electrical conductivity of the subsurface. The measurement is similar to that made by the resistivity method, but is accomplished without ground contact. The EM method permits very high resolution profiling measurements to be made which are particularly effective for locating lateral anomalous conditions.

DIRECT, INDIRECT, AND STATISTICAL APPROACHES

A key to optimizing the subsurface investigation is the balance of indirect sampling and statistical sampling to support data from traditional direct sampling, boring, and lab data. The development of indirect sampling using Near Surface Indicators (NSI) and statistics as an effective aid in subsurface investigation is relatively new. These methods are very useful and certainly cost effective in rapidly converging upon anomalous conditions.

Direct Detection

Direct detection is accomplished by sensing the anomalous condition itself. Trenches, drilling, or remote sensing may be used as appropriate. This approach provides very high confidence levels in subsurface information, but often is costly and, therefore, usually limited to localized evaluations, where high density measurements can be cost-effective.

Indirect Detection

Indirect detection is accomplished by observing indirect evidence of a critical subsurface condition. Often, deeper-seated problems which are active and present potential surface threats will show signs of their presence and activity in the near surface in the form of surface water flow, vegetation stress, paleokarst, or other shallow anomalies. Identification of these Near Surface Indicators (NSI) provides a very rapid means of converging on a potential problem area. It provides an effective means of extrapolating data from the higher-cost direct sampling efforts and still maintaining reasonable levels of confidence. In many cases, the approach has been found to be extremely effective when utilizing the two continuous sampling, surface geophysical methods mentioned previously.

Statistical or Regional Data

In large areas, a geotechnical problem may be the same as in a small area. However, the investigative approach must change. One simply cannot investigate large areas at the same level of coverage and confidence as one can in handling localized problems. Therefore, other approaches must be utilized. The regional assessment problem depends heavily upon integration of a wide scope of information from many sources. These data will provide a regional or statistical overview of conditions. This approach usually does not provide any site specific information, but can provide a very effective means of characterizing large areas in a cost-effective manner. For example, a lineament map may be developed from aerial photography. Such a map would only provide trends over the area of interest, not site-specific details. However, it can be used to guide a more detailed site study.

The applications of these three approaches are summarized in Figure 2. The area to be covered, the type of anomalous conditions expected, the level of confidence desired, and the project budget will determine the approach to be used. The direct approach can be utilized only over limited areas due to costs. While borings may be taken at reduced densities over larger areas, the resulting data is not effective for site specific work, but does provide a data base for intermediate or regional areas. The indirect approach may be applied effectively to provide high confidence level surveys over larger areas and tie-in and extend the results of limited direct sampling. Only a statistical approach or use of generalized regional information may be applied to very large areas beyond the limits of the direct and indirect approaches.

EXAMPLE

An example was selected to illustrate how a broad range of information may be integrated as part of a systems approach and utilized to produce an improved subsurface invetigation of a complex site problem. The assessment includes a regional overview, a local setting assessment, and a site specific investigation. Direct, indirect, and statistical methods were used in the assessment, as were the two continuous geophysical methods: Radar and EM.

The area of concern, less than one square mile, was known to be susceptible to subsidence, sinkhole collapse, and piping. The mission objective was to investigate the potential of subsidence or collapse along a roadway expansion project. Initial on-site drilling revealed that:

- o old sinkholes existed immediately adjacent to the roadway
- o roadway maintenance had been a problem in the immediate area due to localized subsidence
- o considerable clay in-filling and loss of circulation (cavities) or high permeability zones existed

Regional Setting

A regional overview of 10 square miles was made to characterize site conditions. This work was based primarily on state geologic publications and existing aerial photographs. These two sources of information provided a regional/statistical approach to provide an initial understanding of general site conditions. Figure 3 shows the general geologic section obtained from the state publication. Figure 4 shows the distribution of existing sink holes based upon a review of five sets of aerial photography. From the aerial photography, and reports from local cave divers, the lineament trends in Figures 5a and 5b were compiled.

Local Setting

The survey was then focused into the local setting of about one square mile. The lineament trends were used to guide the direction of reconaissance surface geophysical traverses, utilizing both EM and Radar. These two continuous geophysical methods were used to establish more specific locations and trends of fractures in the area. An indirect approach, using Near Surface Indicators (NSI), was used to indicate the presence of fractures, piping, and deeper- seated cavities beyond the range of direct detection by the instruments. Figure 6 shows the location and orientation of shallow fractures in the rock, based upon EM measurements. Figure 7 shows a radar profile of soil conditions; the anomaly is related to a fracture in underlying rock. By carrying out a number of parallel passes, a linear trend could be established in the soil anomaly which correlated with regional lineament trends. Figure 8 shows a large paleokarst feature (sinkhole) which has been filled in with natural sand deposits. Localized piping, related to rock fractures, are also shown. These features remain totally undetected by experienced geologists from surface observations, but are clearly detected by radar.

In addition, a broad-based effort of on-site observations and verbal communications greatly assisted the understanding of site conditions and identified a number of cases of recent subsidence and collapse activity. Statistical information, as well as indirect and direct observations, were utilized in this phase.

Localized Problem Area

The specific site of concern was less than one-tenth square mile. In addition to the initial drilling investigation, subsequently detailed investigation was accomplished by observation, verbal communication, surface geophysics, (EM and Radar), and personal entry in open sinks for underwater observations. The results of EM and Radar traverses from the local setting were then focused at the site-specific area and the major fracture trends could be followed from statistical regional data through the local setting and finally to the immediate site of concern. This "tie-in" to the surrounding area provided a high level of confidence in assessing the fracture patterns and their overall significance to the immediate problem.

The extensive use of on-site observations and personal communication with property owners, utility companies, and local cave divers provided invaluable information regarding location, size, and frequency of subsidence and collapse. In addition, a professional geologist made a dive into an existing open sink adjacent to the roadway to provide first-hand observations as to fracture trends, rock stability, and to assess the overall stability of the sink. Statistical, indirect, and direct methods were used at the local level of the investigation. Finally, direct sampling (drilling) was used to evaluate the extent of fractures (cavities) associated with the indirect results based upon NSI.

RESULTS

The outline of the three levels of investigation is summerized in Figure 9. Starting with an overview of the site setting and incrementally focusing on the local site problem provided the means to rapidly converge upon a technically accurate site understanding in a cost-effective manner. Information from one level alone is usually insufficient to support a highly accurate investigation. As an example, the integration of five sets of aerial photography indicated two-to-five times the number of existing sinkholes than any single photograph. Further, visual observations from walking the site revealed many sinkholes of about three-to-ten meter diameter; hundreds of small one-to-two meter diameter sinks were observed. Virtually, none of the sinks identified by ground observations were detected in the aerial photography.

The results of regional data and limited site specific direct sampling alone are shown in the geologic section of Figure 3. The final composite geologic section shown in Figure 10, was based upon all of the survey data and provides a much higher level of confidence and accuracy.

The level of detail in the final analysis, as well as the levels of accuracy and overall confidence, was very high. The single geologic formation was characterized into three zones:

1) a shallow zone of piping and collapse, where frequent subsidence of one-to-two meters diameter was occuring (Collapse was easily triggered by light construction or surface drainage.)
2) an intermediate zone (possibly an unconformity within the formation) in which extensive loss of circulation occured and whichg could be the originating cause of the shallow activity
3) a deeper zone in which the larger cavity system originates (This deeper zone is the source of major collapse sinks as well as major fracture patterns, as shown in Figure 10.).

All of the large collapses, as shown in figure 10, appear to be old and none of the data indicates potential of large collapse at this time. However, large fracture zones, as shown in Figure 10, were proven to be connected to the surface and showed signs of activity. One site investigated showed a radar anomaly (NSI); when drilled, only sand was found to a depth of 140 feet. This narrow fracture was in direct alignment with major lineaments identified in the Regional Survey.

Because of the correlation that existed between: regional linements, identified in the regional survey; radar linements, identified in the local setting assessment and site-specific evaluation; and drilling conformation; there was great certainty that this anomaly was part of a major fracture system and could be a potential problem.

SUMMARY

A systems approach is necessary to cost-effectively achieve success in investigating complex subsurface conditions. The use of remote sensing and geophysical methods are becoming powerful techniques in the application of the systems approach. More specifically, two contemporary techniques -- Ground Penetrating Radar (GPR) and Electromagnetics (EM) -- with their unique ability to obtain continuous spatial measurements, offer major breakthroughs in geotechnical subsurface investigations. The combination of these two techniques, with other subsurface investiation techniques, results in yet further improvements. The use of indirect detection and "Near Surface Indicators" to locate anomalous conditions permit rapid convergence on local problems.

Clearly, there is an increasing understanding of the benefits of utilizing non-destructive in-situ remote sensing or geophysical methods in subsurface investigations. However, the benefits of continuous data still are not totally appreciated by most site investigators. Indirect measurements and use of near surface indicators (NSI), as well as a statistical approach, are just beginning to be used.

The approach and methods presented are not necessarily a panacea in themselves, but tied together in an integrated but flexible systems approach, which is properly implemented, they become a very powerful means to signficantly improve the accuracy of subsurface investigations.

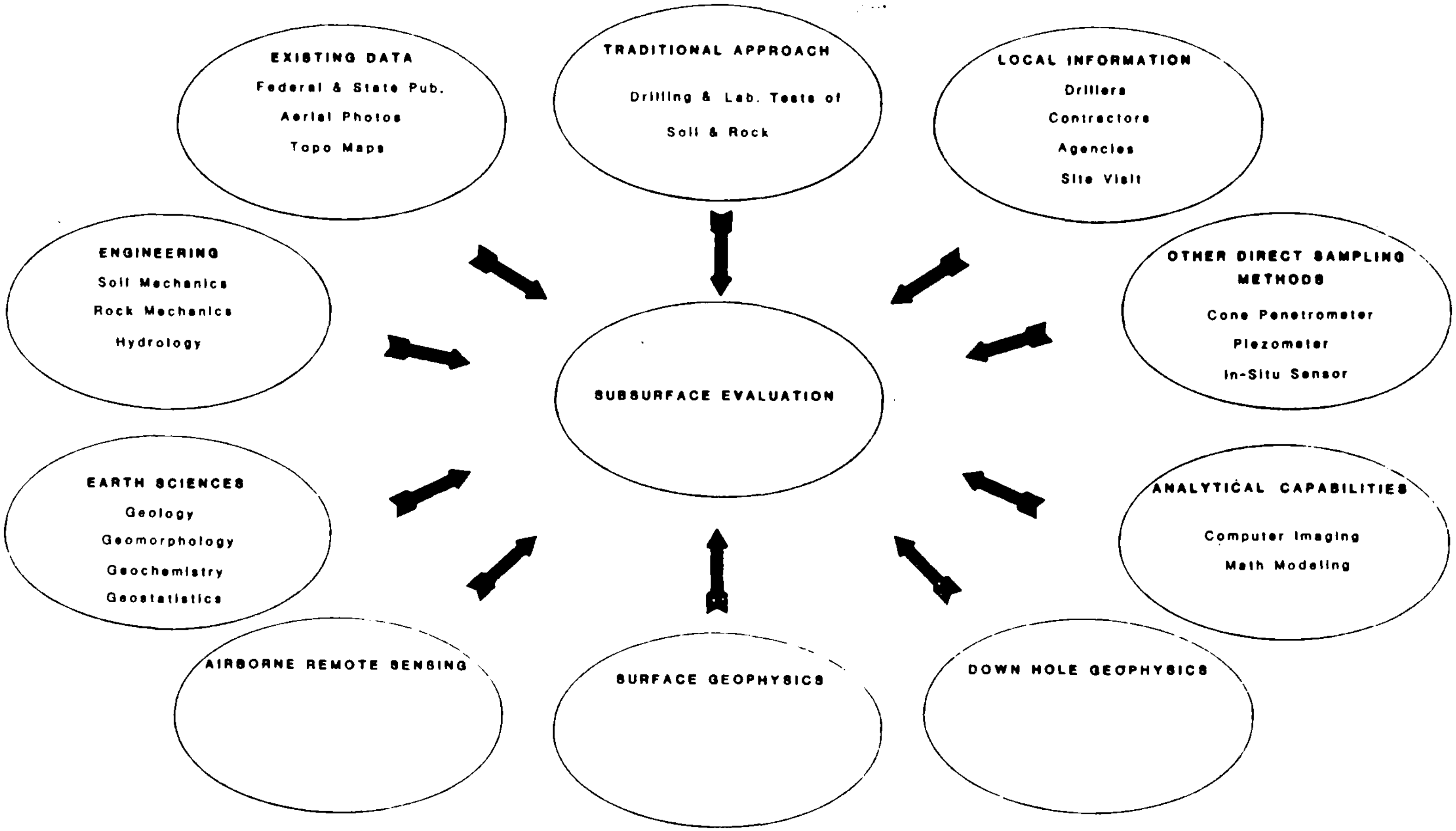


FIG.1 Technical resources and tools which can be utilized in a subsurface investigation.

APPROACH	LOCAL (.1 sq. mi.)	INTERMEDIATE (1 sq. mi.)	REGIONAL (10 sq. mi. or more)
DIRECT	primary	secondary	secondary
INDIRECT	primary	primary	secondary
STATISTICAL	primary	primary	primary

FIG.2 Application of direct/indirect and statistical approaches to site investigation

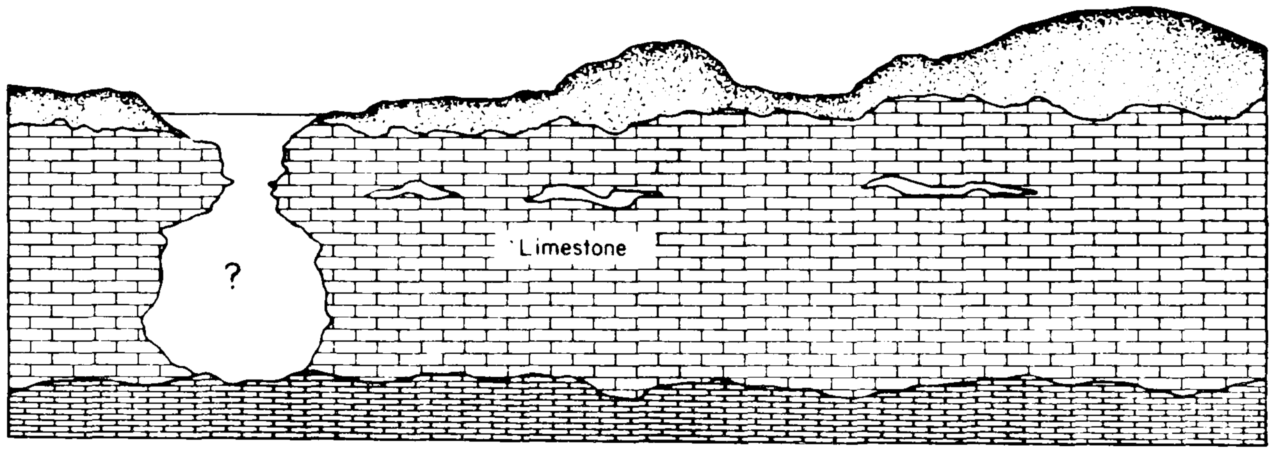


FIG.3 Generalized geologic section based upon regional information and initial on site drilling.

FIG.4 Distribution of sinkholes based upon existing aerial photography.

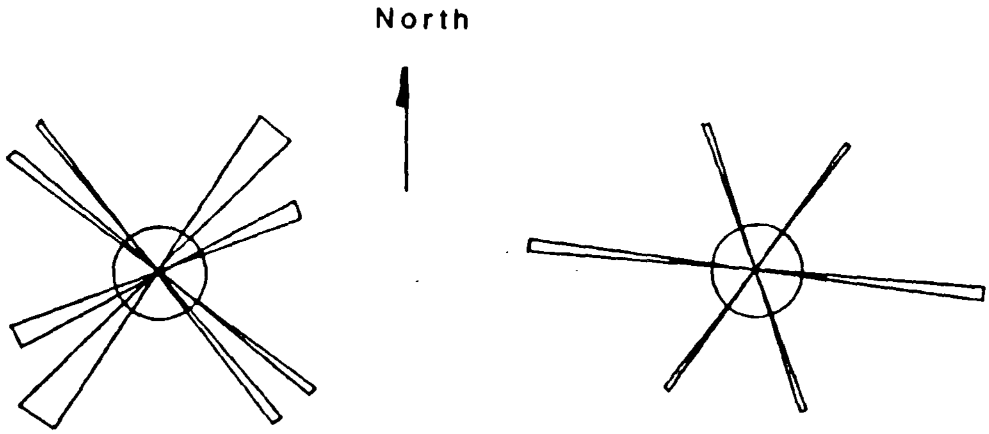


FIG.5 Lineament angle relationships.

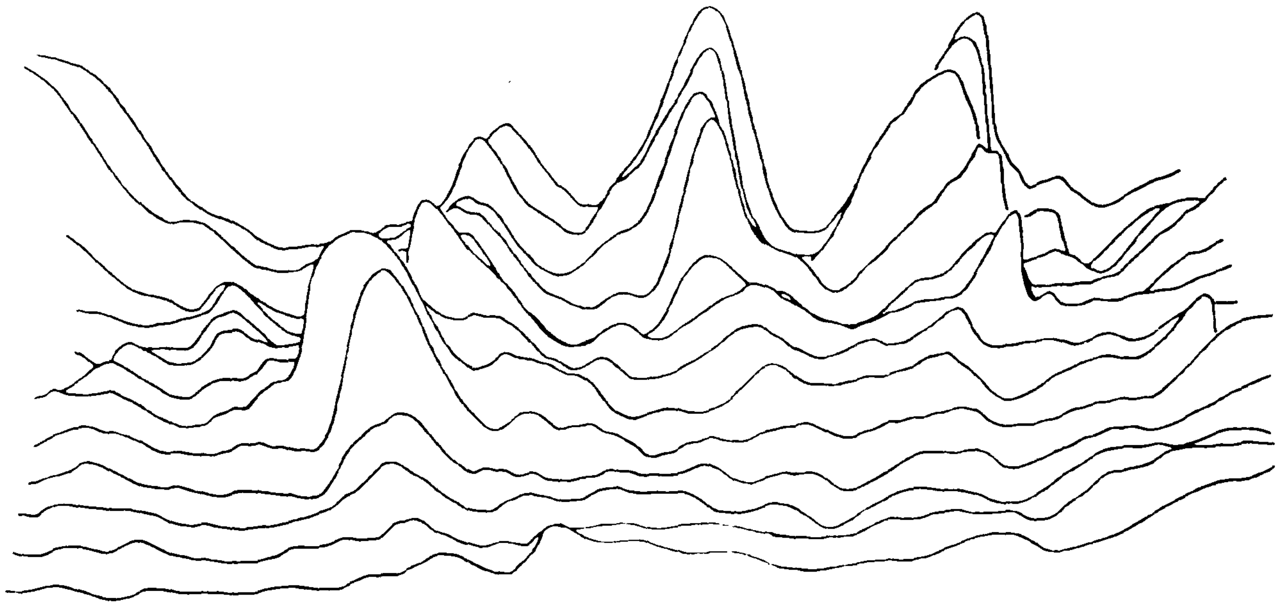


FIG.6 EM conductivity data shows fracture trends in rock.

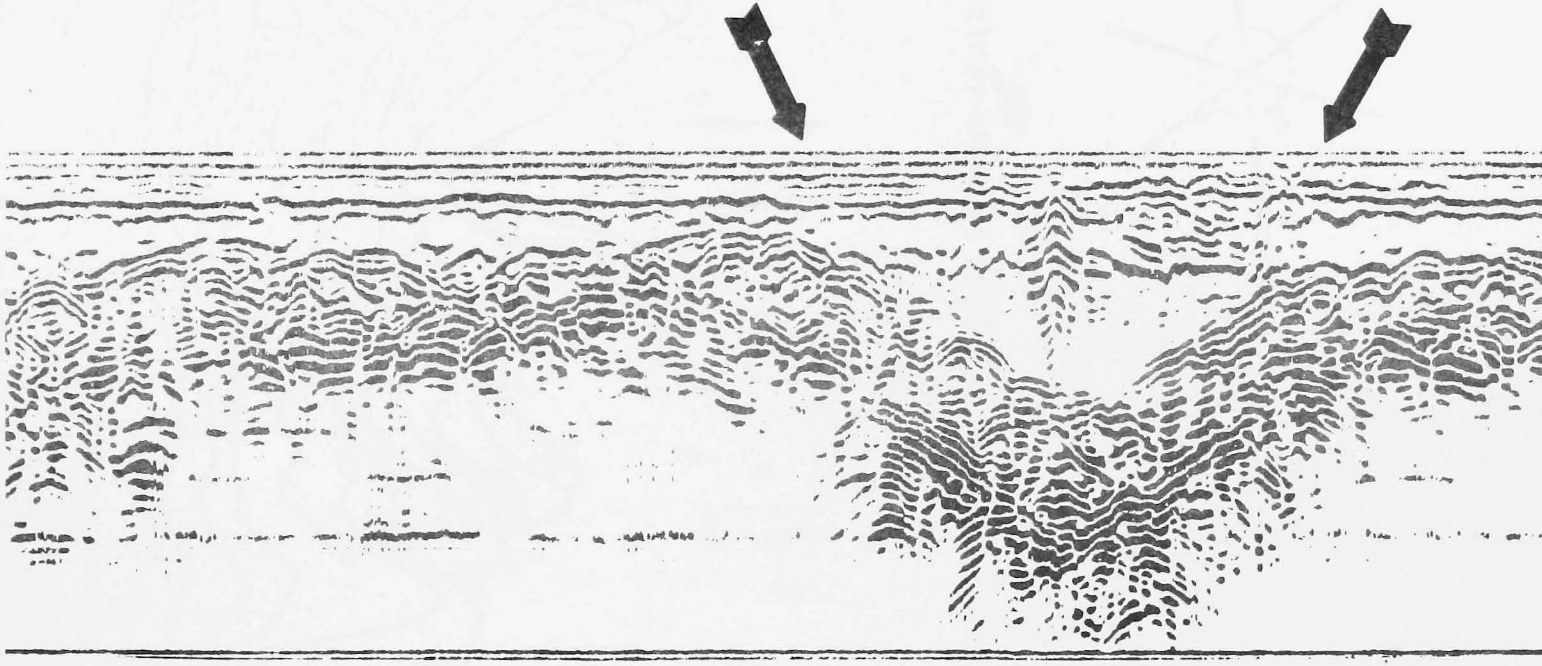


FIG.7 Radar data shows anomalous soil conditions related to underlaying fractures in rock.

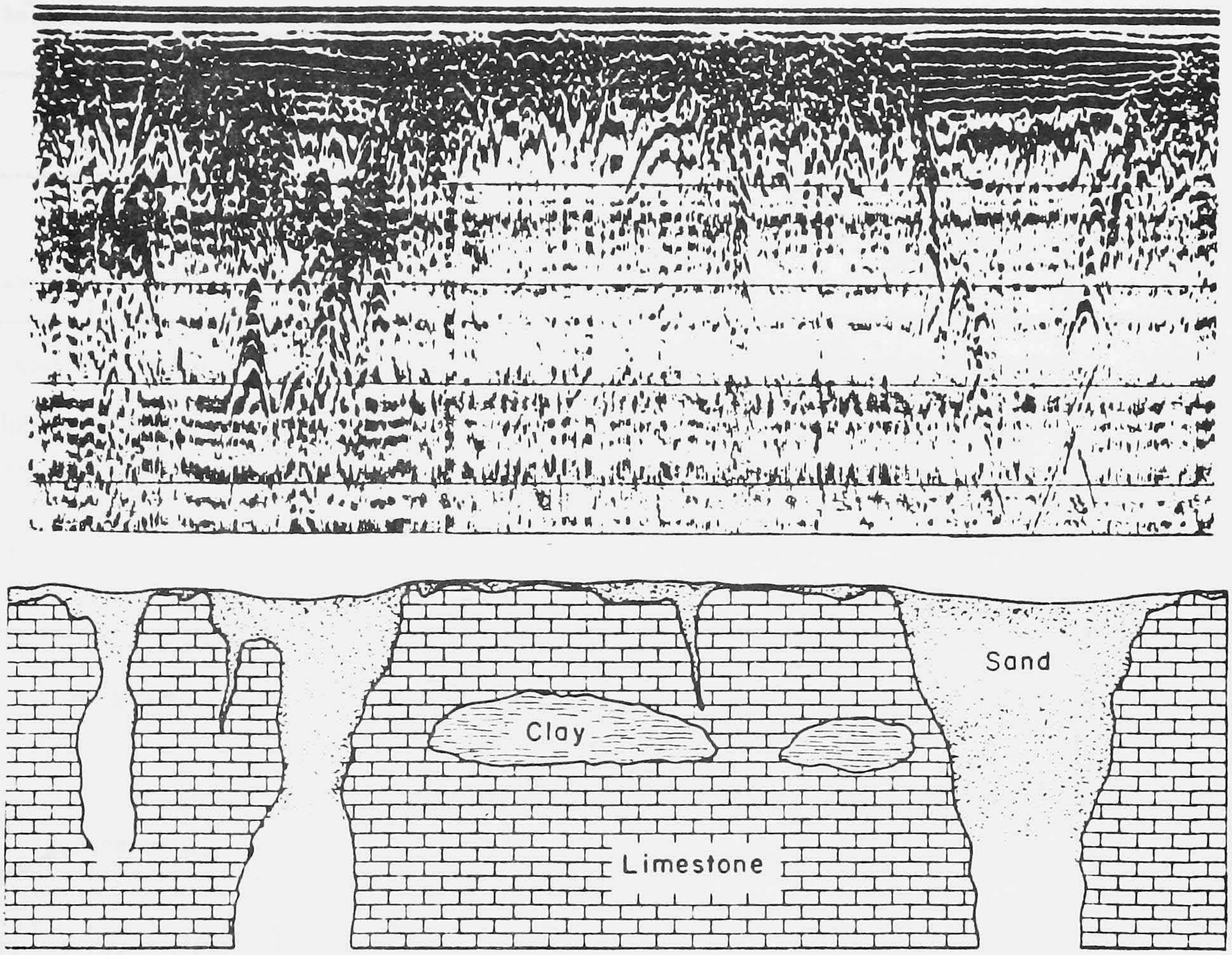


FIG.8 Radar data and generalized cross section showing large sinkholes and piping.

	REGIONAL SETTING	LOCAL SETTING	LOCALIZED PROBLEM AREA
SIZE	10 Square Miles	1 Square Mile	.1 Square Mile
OBJECTIVE	Characterize the general setting	Improve information on general setting. Converge upon local anomalies	Provide detailed character of local geology, to evaluate stability.
APPROACH	Regional/Statistical	Indirect/Near-Surface-Indicators Statistical Fracture Trends	Direct and Indirect Near Surface Indicators
SOURCE OF INFORMATION	o State Geologic Pub. o Aerial Photos, (5 sets) o Cave Divers	o On site observations o Verbal Communications with land owners and utilities. o Surface Geophysical Data (EM/Radar)	o Detailed site specific observations. o Drilling o Surface Geophysics (EM/Radar
RESULTS	o General understanding of geologic setting. o Some characteristics of sinkhole history, number, and location. o Lineament maps of "fractures from aerials o Gross character of larger cavities from cave divers.	o Greater number of sinks identified. o Numerous small local collapse problems. o Tie in regional picture to local setting and identify location of fracture trend	o Piping connected with deep cavities. o Major cavity system is stable. o Small shallow piping is very active.

FIG.9 Overview of objectives and results from example.

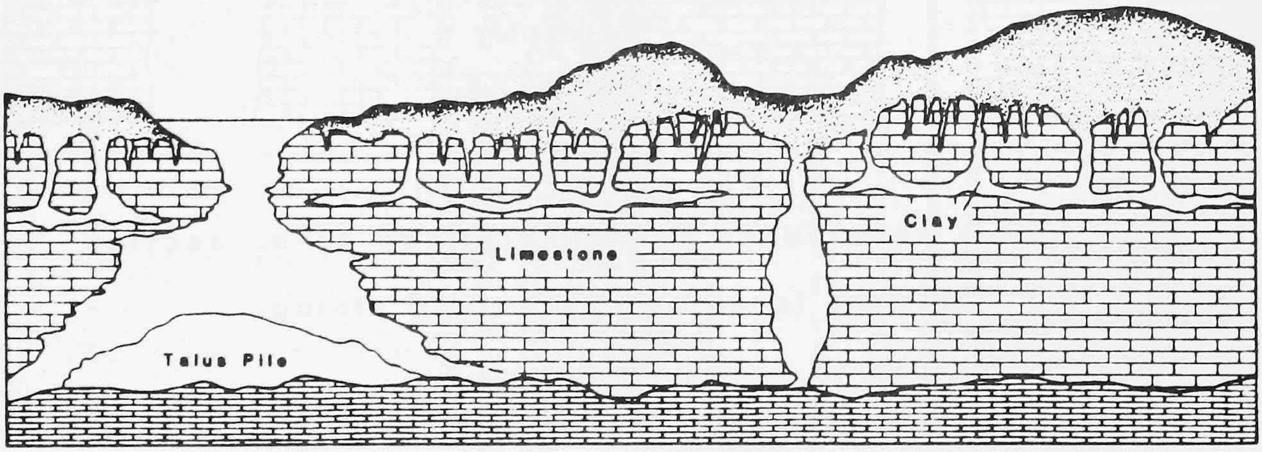


FIG.10 Generalized geologic section based upon integration of all information derived from example project.

Geotechnical Applications in Maintenance and Reconstruction of the Existing Highway System

by

William F. Sherman
Wyoming State Highway Department

Abstract

With the near completion of the Interstate System, environmental problems in new highway locations, and the energy shortage, the engineering geologist and geotechnical engineer face a new challenge. This is application of their expertise in the maintenance and reconstruction of the existing system.

The maintenance and reconstruction phase of the highway program will provide the engineering geologist and the geotechnical engineer with the opportunity to examine the results of their past work. It should allow us to determine what we have done right and wrong and learn from these experiences.

The purpose of this paper is to discuss the changes in attitudes and approaches considered necessary to provide the type of geotechnical information useful in this phase of the highway program. It is planned to discuss things we in the Engineering Geology Branch of the Wyoming Highway Department have been doing in regard to maintenance and reconstruction.

INTRODUCTION

In the past 25 years the primary emphasis in highway geology and geotechnics has been on highway location, design, and construction. This was the result of the national goal to create a new highway system for both defense and transportation requirements of the country, which was the interstate program funded by the highway trust fund.

Since the emphasis has been on "new construction", the majority of the technical papers and research have been concerned with this phase of highway development. In fact, in any geotechnical literature review it has been very difficult, if not impossible, to find technical papers directed toward maintenance and/or reconstruction of existing facilities.

The emphasis in the highway industry has now changed. Some of the reasons for the change are the near completion of the interstate system, environmental considerations, and the high cost of fuel. The primary direction now is on maintenance and/or reconstruction of the existing system. The change in thinking in the industry is illustrated by the subjects discussed at such meetings as Western Association of State Highway and Transportation Officials and publications of the industry in general.

The new funding categories employed by the Federal Highway Administration such as the "4R" program (Resurfacing, Rehabilitation, Restoration, Reconstruction) is indicative of the direction of the industry. The funding for these categories has increased significantly over the past years and may well be the principal categories in future FHWA programs.

We, as geotechnical professionals in the transportation field, must also be changing our thinking. If we are to meet the challenges, our approach to the new problems must be updated. The application of geotechnics to the maintenance and reconstruction programs presents us with one of our greatest challenges.

PRESENT TRENDS IN THIS GEOTECHNICAL AREA

The primary change, it is felt, should be in attitude. Certainly, we will be utilizing our past training and experience; however, our approach will be somewhat different. For example, in the maintenance phase availability of funds will be much more restricted than we have been accustomed to in the past. Solutions of geotechnical problems are much more challenging and demanding when working within a very restrictive budget constraint than when the cost is secondary.

This situation is not new in our profession as indicated by several of the past presentations of the various Highway Geology Symposiums. However, these papers are certainly the exception rather than the rule when you consider the profession as a whole.

A typical example of this situation is the emphasis placed on the work being done with clay shales, one of our principal problem creators. Numerous excellent published reports have discussed methods of identification of these shales, and have detailed information on specifications to be used in new construction to reduce the problems created when these shales break down with time in embankments. To my knowledge there has been no published data on how to correct these problems on existing embankments, which is now the critical situation.

Another example of the emphasis within our profession is the recent draft copy of the revised Manual on Foundation Investigations. Approximately four years ago the National Cooperative Highway Research Program was given the task of updating and improving the manual on foundation investigation by the American Association of State Highway & Transportation Officials committees on Materials and Bridges. The document produced was two volumes, 15 chapters, and over 400 pages. It is probably the most complete compilation of information on subsurface investigations to date. However, although the manual includes the types of investigations, and in-depth discussion of the information to be obtained, there is not a word concerning the data to be obtained from the observation of existing conditions, i.e. (1) misalignment of bridge rail indicating substructure settlement (Figure 1), (2) heaves indicating swelling shales (Figure 2), (3) roadway settlement indicating collapsing soils (Figure 3). These are only a few of the types of information that should be a major part of a manual on subsurface investigation. This truly illustrates where our thinking is.

In fact, it is estimated in the past year a major part of the subsurface investigations done by the geotechnical people in the state highway department have involved bridge replacements and reconstruction and maintenance of existing roadways. This situation is definitely going to increase in the future.

SUGGESTED CHANGES IN THE GEOTECHNICAL APPROACH

There will be opportunities in this new phase of maintenance and/or reconstruction that we have not had before. These will include the opportunities to see what has really happened within embankment foundation (See Fig. 4 & Fig. 5); how applicable was our design to the real problem; and were the interpretations we made valid or not. This opportunity should provide the needed information to improve our interpretations of subsurface conditions and reduce the safety factors and thus the cost in future geotechnical designs.

A change in the methods of application of geotechnics to the maintenance and reconstruction phase will be required. For example, it is proposed that the major geotechnical tool in this phase is monitoring. The greatest need in maintenance is to detect factors which are providing the conditions for failures and to interpret or stop this trend and avoid the failure. At the minimum, the need is to retard the progression involved in foundation failures and to provide the opportunity to correct the overall situation at a more optimum time.

The types of monitoring suggested include aerial photos taken periodically of the same areas, ground photos taken periodically of the same areas, on-site inspection, and instrumentation.

Aerial photos taken of potential problem sites provide the best monitoring device available. The trained photo geologist can (a) detect potential failure in time for preventative maintenance, (b) outline areas which should be instrumented to determine the extent and type of failure, and (c) obtain data for design of future corrective measures. An additional use of the photos is as illustrations for explaining problem areas to engineering personnel. The above examples are only some of the uses for aerial photography in this phase.

Ground photography also can be used as a supplementary monitoring tool. The ground photographs can be used to review observations made on aerial photos. Normally, this type of photography is taken for use by the Planning Division and is readily available at no additional cost to the geotechnical personnel.

The on-site inspection of potential failures is a third method of monitoring. The area reviewed is restricted compared to the aerial photo coverage; however, this phase of the monitoring is necessary to verify interpretations made in the photo study. The relationship is the same as the steps utilized in any photo geologic mapping project.

The fourth monitoring phase is instrumentation. Instrumentation is the most expensive monitoring method and should be used only when progressive failure is occurring or where failures can jeopardize human lives. Insturmentation is an indispensable tool in obtaining the data required for correcting stability problems.

EXAMPLES OF A MONITORING SYSTEM

An ongoing monitoring program is the "Slide Monitoring Program" presently being conducted by the Geology Branch of the Wyoming Highway Department. The following steps are followed in establishing and maintaining this program:

1. Potential problem areas on the existing highways are established, based on geologic condition, geotechnical experience, and input from the field maintenance personnel.

2. Colored aerial photos are taken of these areas annually.

3. A field review is made of the sites which indicated potential problems based on the study of the aerial photos.

4. Following the field review, which included input from the maintenance personnel involved with the particular site, instrumentation is installed.

5. Annually a report is prepared by the Geology Branch and submitted to each District Maintenance Engineer. The report includes (1) which areas appear to be the most critical, (2) recommendations on remedial measures which can be accomplished by maintenance forces, (3) annual maintenance costs on each critical area, (4) areas recommended for instrumentation, and (5) recommendations on reconstruction measures on areas that have failed or are in the process of failure.

Annually new areas are added and stabilized areas omitted. Any sites which have been repaired are monitored for at least two years before they are taken off the program.

This procedure has provided information for prevention of failures plus reduced significantly the emergency type failures. The cost of the program has been recovered in the information provided in design of correction methods for stability failures.

An example of the use of aerial photos used to monitor a stability problem in this program is shown in Figure 6 through Figure 10. These are stero pairs to aid in the photo study.

Figure 6 is a photo taken of a potential stability problem in 1974. Note on the photo areas marked A, B, and C. The progress of the conditions at these locations is illustrated in aerial photos taken every two years. In Figure 7 Areas A and C show only minor movement. Tension cracks can be seen to start to develop in Area B indicating movement taking place.

Figure 8 taken in 1978 shows no new movement in Area C where failure had occurred prior to 1974 and one would expect progressive failure. Area B shows continued movement and progressive failure toward the roadway. Area A shows only minor movement at the head but extensive movement in the toe area.

Figure 9 taken in 1980 shows that Area C is stable, Area B has required maintenance wirk, and Area A with movement only at the toe and not directly affecting the roadway.

Figure 10 taken in 1982 is to illustrate the results of our corrective measures. These measures included underdrains, removal of the slide material in Area B, and replacement with stable material and correction of the toe failure of Area A by providing a berm across the large drainage.

This site will be monitored by annual aerial photos for approximately three years to determine the adequacy of the corrective measures.

SUMMARY

The need for geotechnical information is just as great in the maintenance and reconstruction phase as in the location phase of highway systems. Our attitudes and approach will have to change to meet this new challenge. It is proposed that monitoring can help to provide the necessary information which will be required for the solution of the geotechnical problems incumbent in the maintenance and reconstruction phase of the Transportation System.

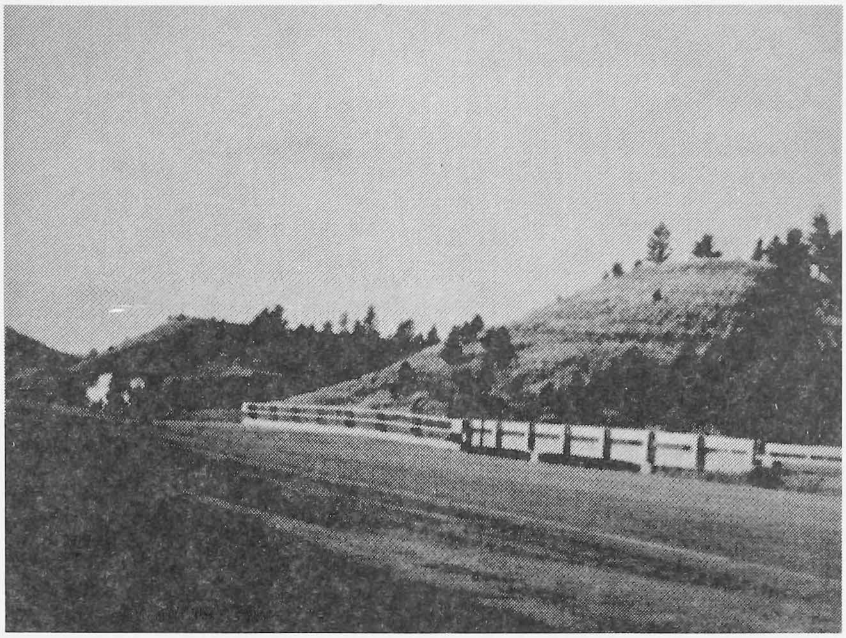


Fig. 1: Note the sag in the bridge rail in behind the pickup truck. I-90 Northeastern Wyoming.

Fig. 2: Heaves due to a swelling shale. On I-25 (a) North Central Wyoming.

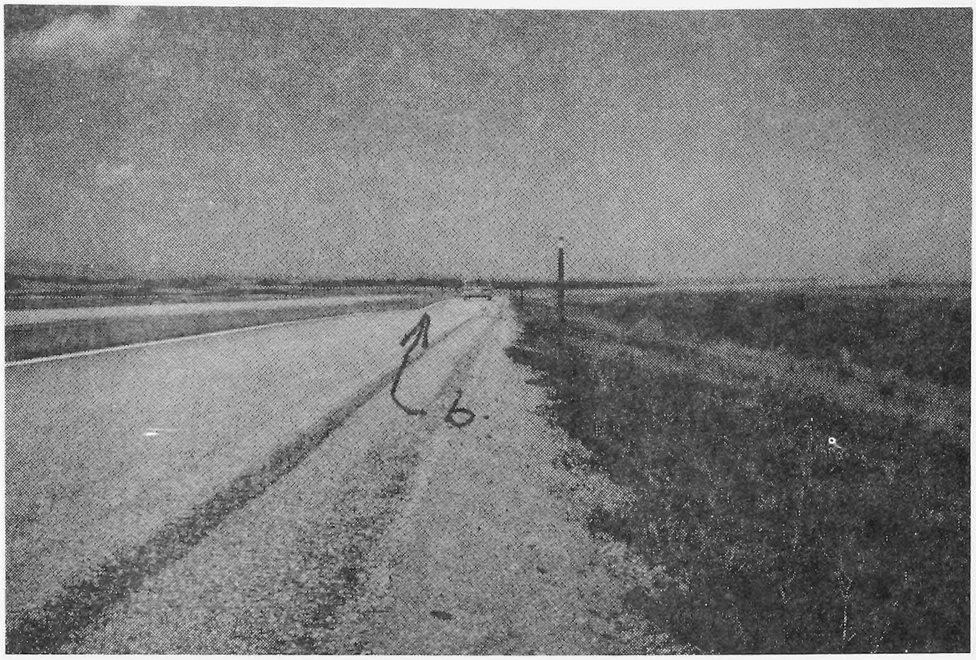


Fig. 3: Settlement failure due to collapsing soils derived from the tertiary shales. I-25 North Central Wyoming.

Fig. 4: Heave in the shale bedrock which is the foundation for the roadway. Heaving occurred due to concentration at the surface water by the roadway section.

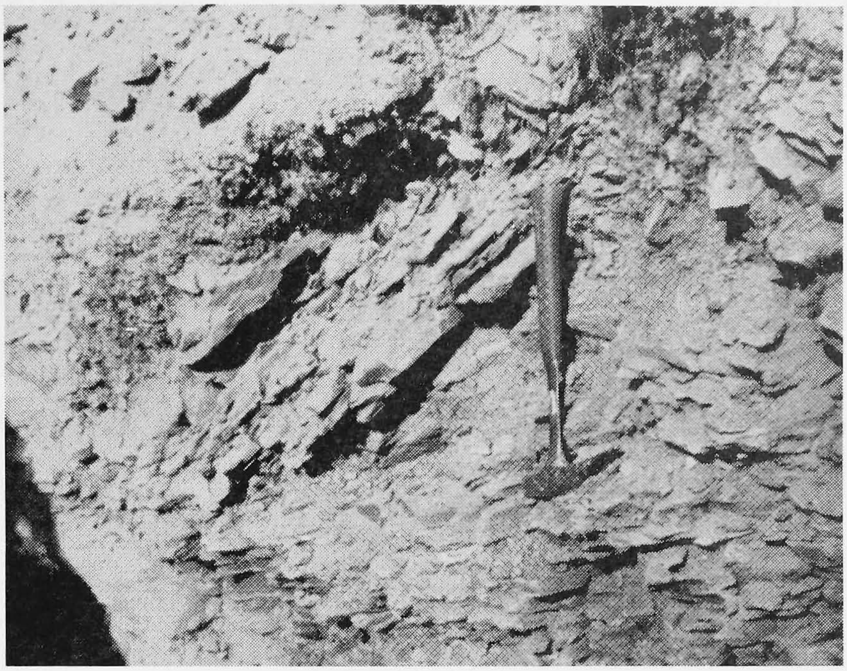


Fig. 5: Heaving in shale below the existing roadway. Note the horizontal beds approximately four feet below the heaved section.

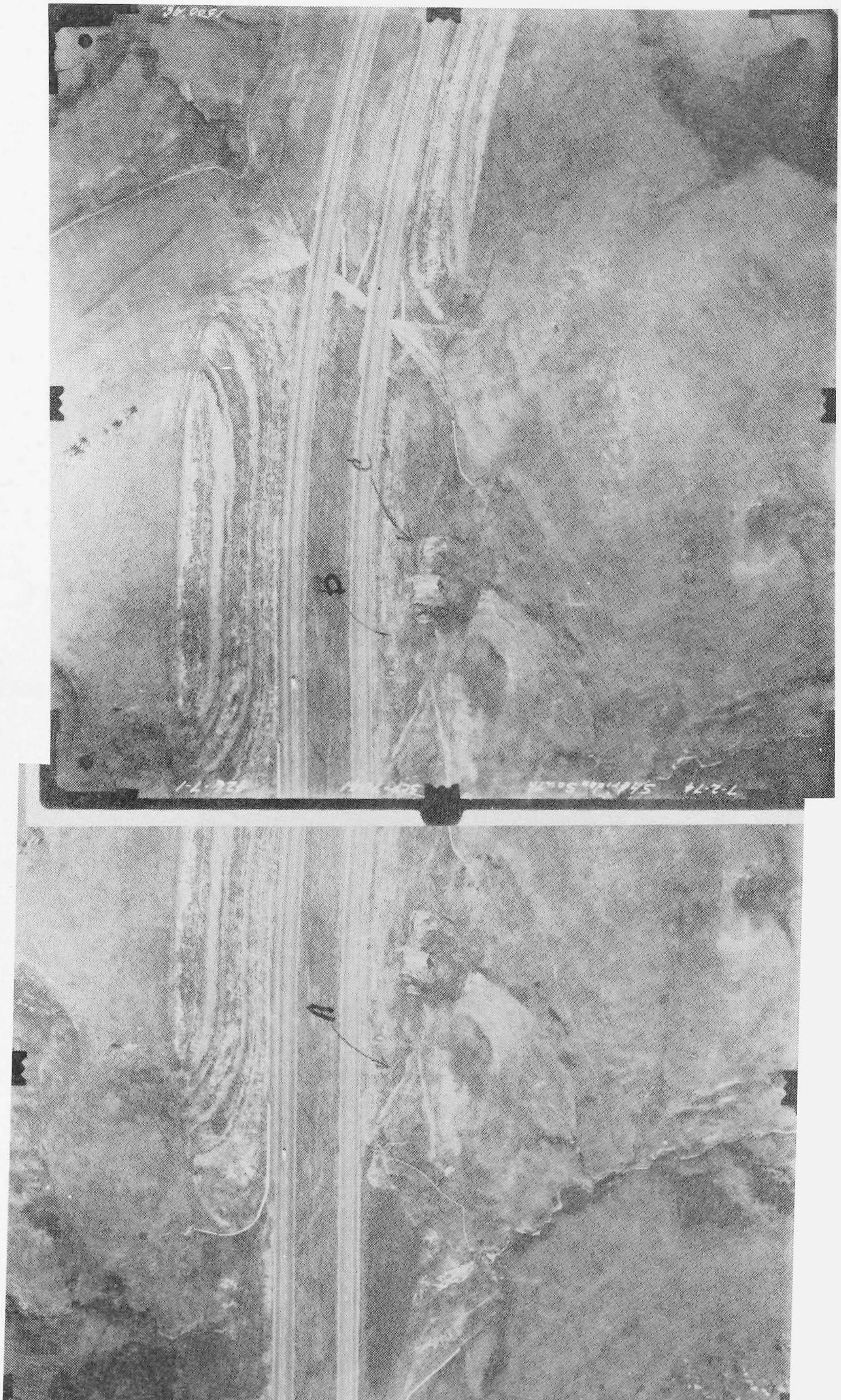


FIGURE 6

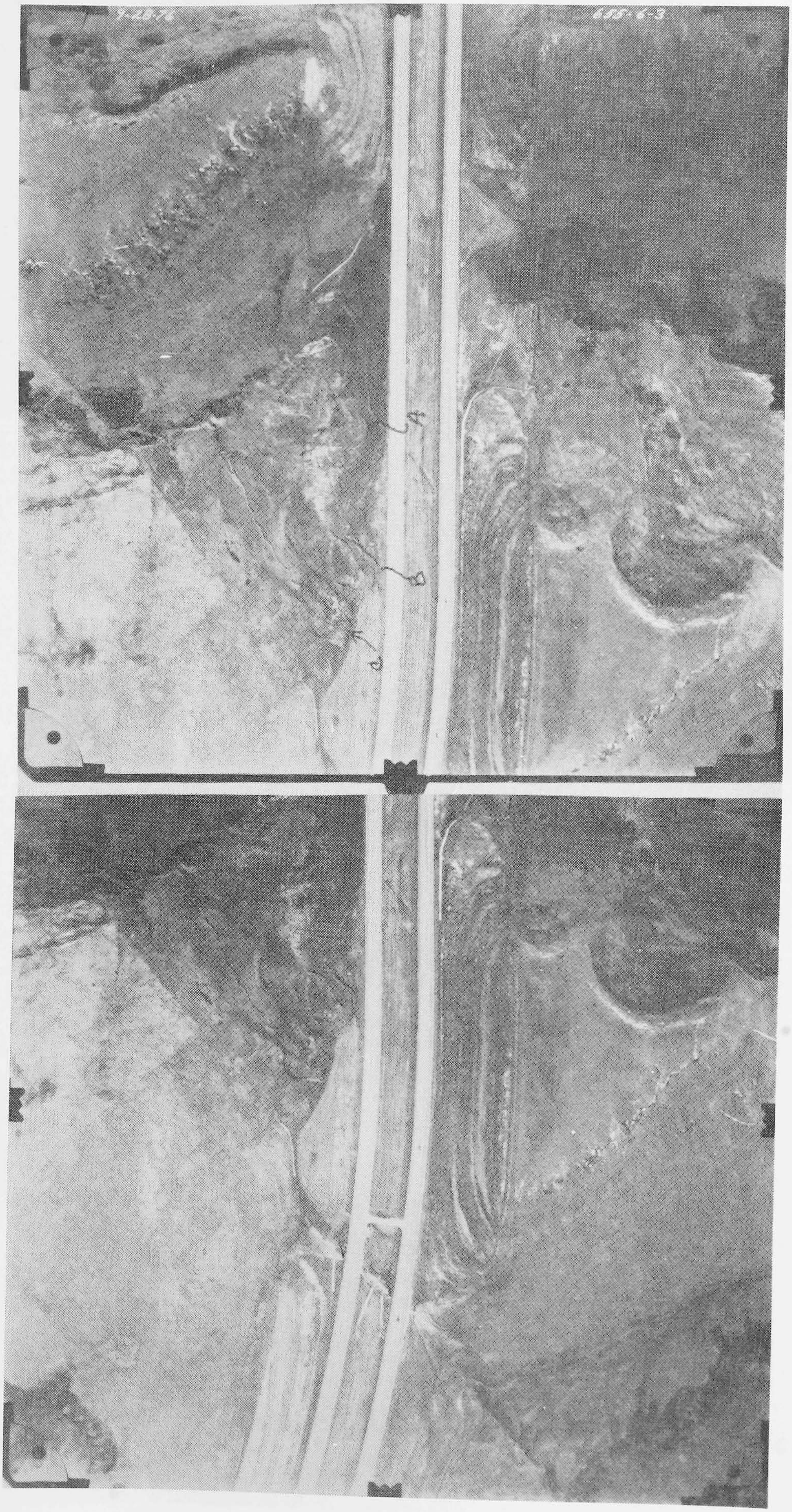


FIGURE 7

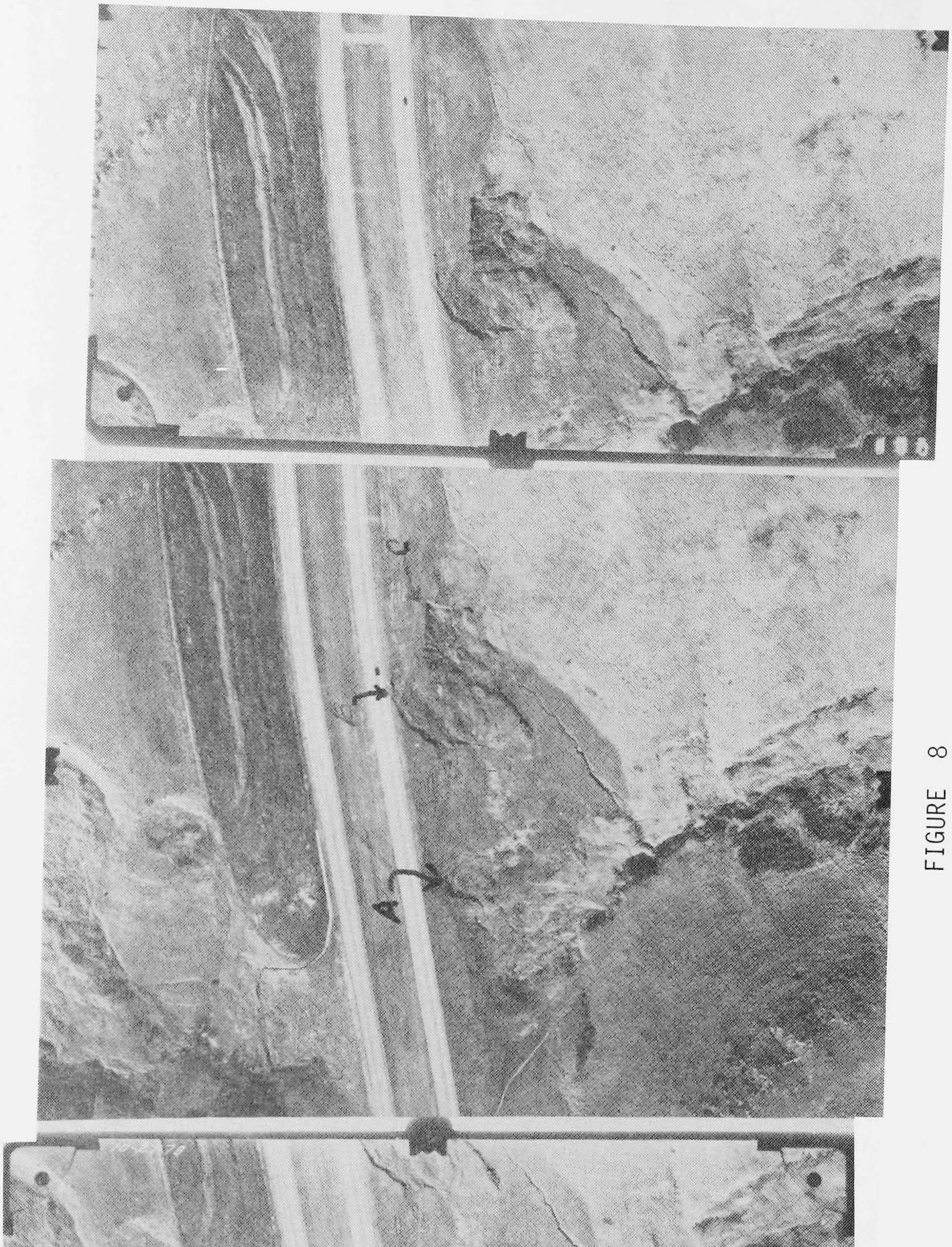


FIGURE 8

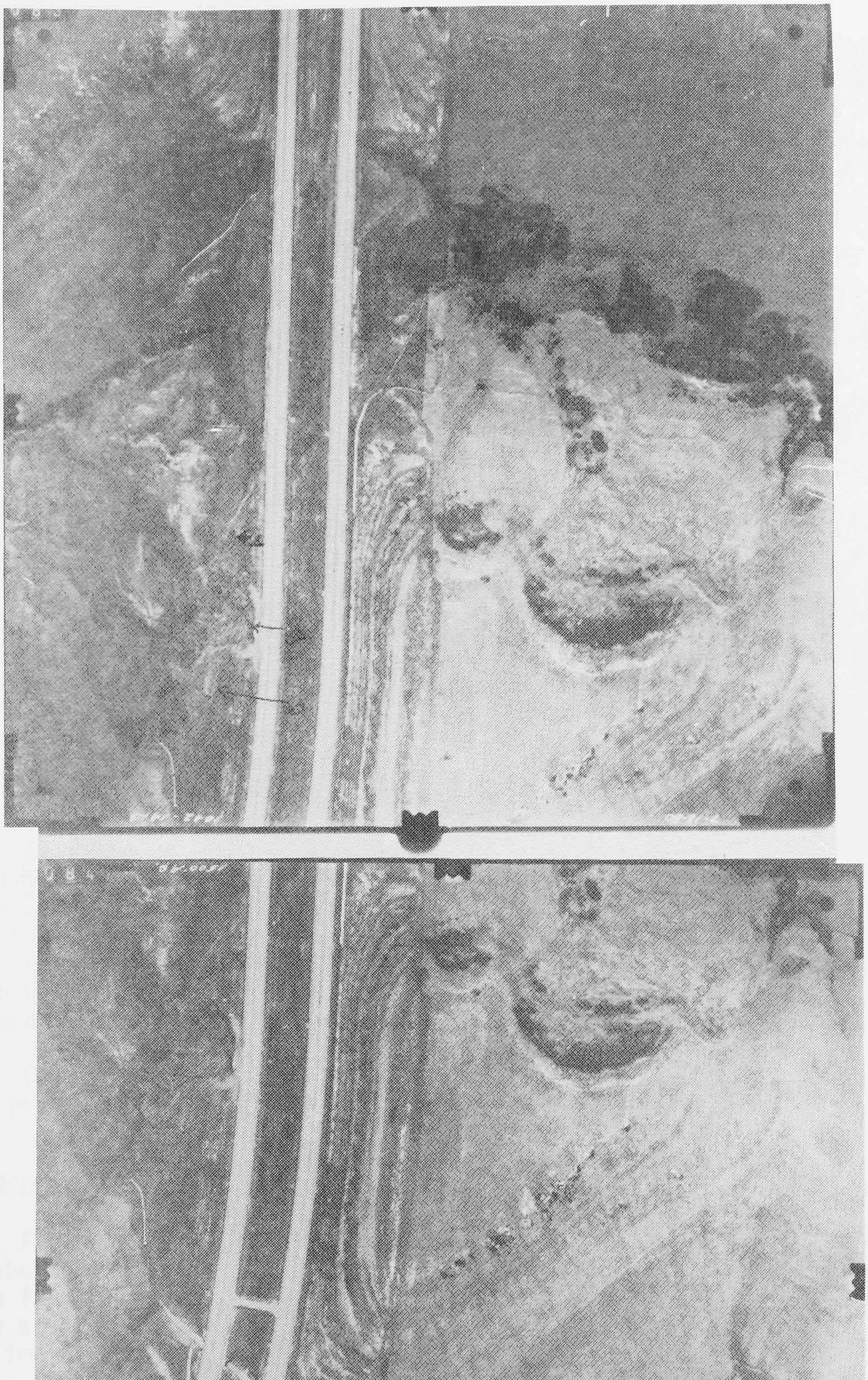


FIGURE 9

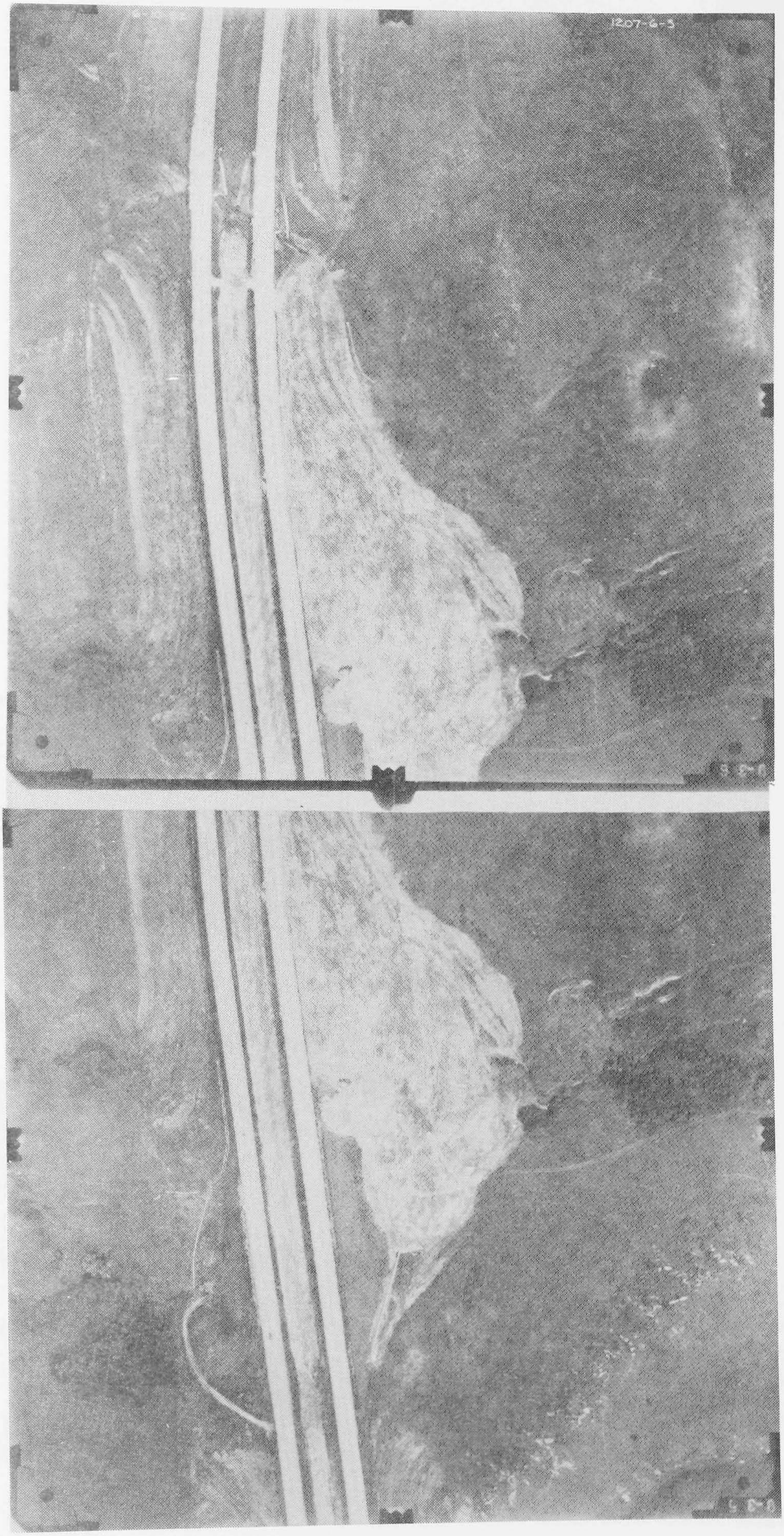


FIGURE 10

FLY ASH LEACHATE IN HIGHWAYS

By

Sam I. Thornton
Department of Civil Engineering
University of Arkansas
Fayetteville, AR 72701

ABSTRACT

Fly ash, a pozzolanic by-product of coal burning, can replace lime or portland cement in highway base or fill construction. Only 13.3% of the fly ash produced in 1980 was used in commerical applications, making fly ash an abundant source of highway construction material.

Leachate from a fly ash fill is a potential environmental problem. Effluent from tests on an ASTM type C and type F fly ash produced from Wyoming and Kansas coals had high pH, alkalinity, and dissolved solids concentrations. The pH ranged between 10.7 and 11.8 for both fly ashes depending on the type and concentration. Alkalinity reached a maximum of 580 mg/l and hardness 640 mg/l. All values decreased with increasing volume of water. Some fly ashes from Michigan reported pH's between 7.4 and 11.4 with total dissolved solids near 1700 mg/l.

Permeability of fly ash in fills varies widely. When placed in a slurry, the type C fly ash from Wyoming has a permeability of 3×10^{-6} cm/sec. Even when mixed with sand or clay on a 50% by weight basis, the soil-fly ash mixtures had permeabilities in the 10^{-6} cm/sec range. When placed dry, however, the type C fly ash developed cracks due to the reaction with water, creating secondary permeability of 1×10^{-3} cm/sec. Observed permeabilities of the Michigan fly ashes were in the 10^{-3} cm/sec range.

Covers over fills to prevent infiltration or encapsulation of bases can prevent the formation of leachate.

INTRODUCTION

Fly ash, a pozzolanic by-product of coal burning power plants, is an abundant potential source of highway and embankment construction material. Some fly ashes are suitable for use as a supplement or replacement for lime and portland cement in soil stabilization applications. Production of lime and portland cement requires heat and will become more costly as energy costs rise. Fly ash, however, is a by-product of power production.

Production of fly ash in the United States was 48.3 million tons in 1980 (National Ash Assn., 1982). The total of fly ash utilized in 1980 was 6.4 million tons or 13.3% of production, making fly ash an abundant source of highway construction material. The unused fly ash is wasted either by sluicing to ponds or hauling to solid waste disposal areas. Disposal operations are quite expensive and require the use of land which could be used for other purposes.

Highway applications of fly ash, however, must consider possible effects on the environment. The leachate from a fly ash fill is a potential environmental problem.

FLY ASH

The properties of fly ash are a function of coal source and preparation, boiler design and firing conditions, and fly ash handling and storage methods (Meyers, et al., May 1976). A high degree of variability can occur in fly ashes, not only between power plants, but within a power plant.

Fly ash is silt sized and spherically shaped. Sixty to eighty percent by weight of particles usually fall between the No. 200 sieve and clay sizes (.074mm to .002mm).

Chemical composition of fly ashes is largely dependent on coal source. Most fly ashes fall into two major groups; those produced from bituminous coals primarily from the eastern United States and those produced from sub-bituminous or lignite coals primarily from the western or southern United States. Typical eastern ashes are high in silica, aluminum and iron oxides. Figure 1 contains the sum of analyses on twenty-five fly ashes from Kentucky (Rose, et al., February 1979). Typical western or southern ashes are higher in calcium and sulfur oxides, usually above ten percent by weight. Table one contains the chemical properties of a typical sub-bituminous fly ash.

LEACHATE

Effluent on an ASTM type C and type F fly ash produced from Wyoming and Kansas coals had high pH, alkalinity, and dissolved solids concentrations (Reed, 1976). For fly ash concentrations of .1% after an hour the type C ash had a pH of 11.3 and the type F ash had a pH of 10.7. At concentrations of 1% after an hour the type C fly ash had a pH of 11.8 and the type F ash had 11.5. At high concentrations, the pH approaches the pH of free lime. Alkalinity increased with increases in fly ash concentration and contact time. At 30 minutes, a 1% concentration of type C ash produced a total alkalinity of 380 mg/l as $CaCO_3$ and the type F ash 200ppm. The maximum alkalinity produced was 580 mg/l (Parker and Thornton, 1977). Total hardness also increased with fly ash concentration and contact time. At 30 minutes, a 1% concentration of type C ash produced a total hardness of 510 mg/l as $CaCO_3$ and the type F ash 260 mg/l. The maximum hardness produced was 640 mg/l. All values of pH, alkalinity and hardness decreased as the concentration was decreased by increased water volumes. Some fly ashes from Michigan had pH's between 7.4 and 11.4 with total dissolved solids near 1700 mg/l (Table 2) (Gray and Lin, April 1972).

PERMEABILITY

Permeability of fly ash in fills varies widely. When placed in a

Table 1

Chemical Properties of
Typical Sub-Bitimious Fly Ash

Compound	Chemical Composition % by weight
SiO	34.0
Al_2O_3	13.0
Fe_2O_3	6.0
CaO	20.0
MgO	6.0
K_2O	0.8
Na_2O	2.8
SO_3	13.7
TiO_2	1.0
Undetermined	2.7

Table 2

Chemical Analysis of Leachate
From Michigan Fly Ashes

	Marysville	Trenton Channel	Karn	Cobb
Ca^{++}	200	288	245	856
Mg^{++}	266	402	357	184
$SO_4^{=}$	230	220	200	-
Boron	0.3	0.4	0.4	10.0
TDS, mg/l	1029	1448	-	1697
TDS, % of Dry Solids	1.0	1.4	-	1.7
pH	11.4	7.4	10.8	11.2

from Gray and Lin; April, 1972

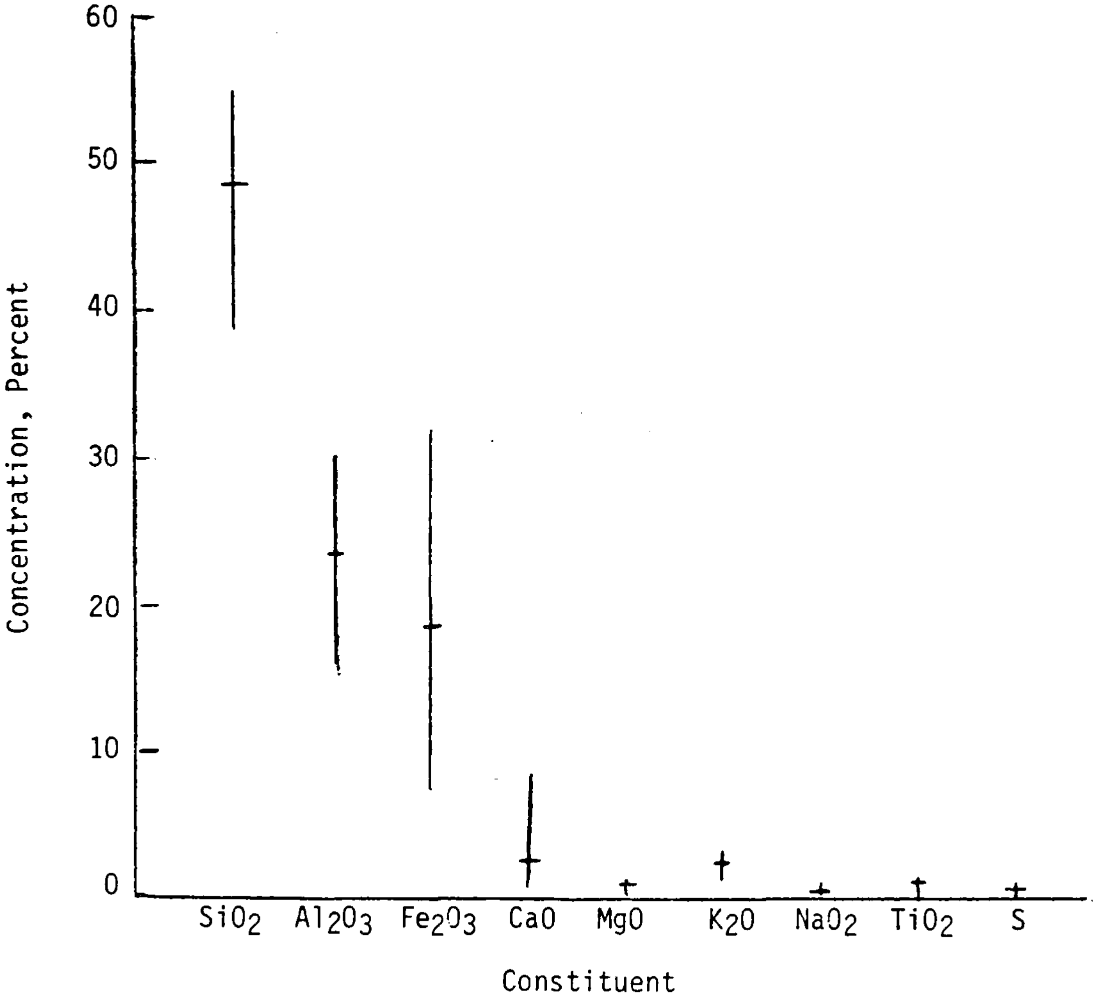


Figure 1. Chemical Constituents of Kentucky Fly Ashes (From Rose, Lowe, and Floyd, 1979)

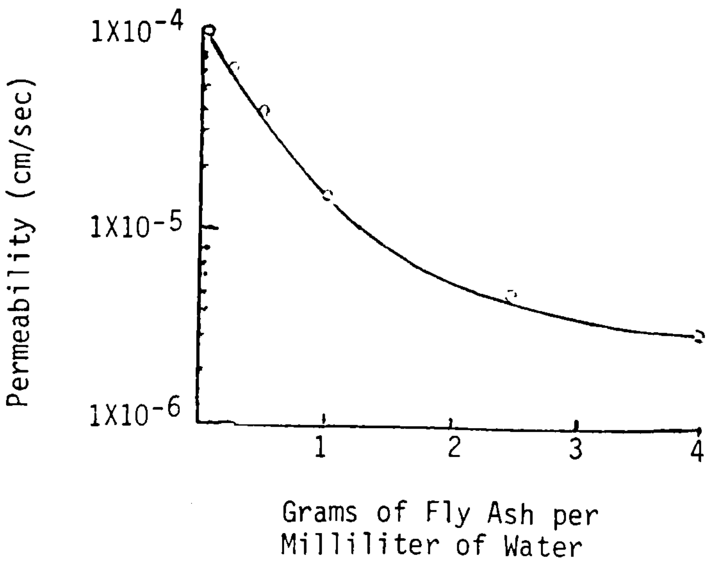


Figure 2. Permeability of Fly Ash - Water Slurry Mixtures

slurry, a type C (sub-bituminous) Wyoming fly ash had an ASTM D-2434 permeability between 5.8 x 10^{-5} cm/sec and 3.2 x 10^{-6} cm/sec (Figure 2). When mixed with sand or clay on a 50% by weight basis, the mixtures had permeabilities in the 10^{-6} cm/sec range. When placed dry, however, the type C Wyoming fly ash developed cracks due to the chemical reaction with water, creating a secondary permeability of 10^{-3} cm/sec. Observed permeabilities of the Michigan fly ashes were in the 10^{-3} cm/sec range (Gray and Lin, April 1972).

Formation of leachate may be prevented by providing covers over fills or encapsulation of bases.

REFERENCES

Gray, Donald H., and Yen-Kuang Lin, April 1972, "Engineering Properties of Compacted Fly Ash," ASCE JSMFE SM-4, p. 361.

Meyers, James F., et al., May 1976, "Fly Ash as a Construction Material for Highways," U.S. Department of Transportation, FHWA-IP-76-16.

National Ash Association, May 1982, "Ash Production and Utilization Survey," National Ash Association, Suite 510, 1819 H Street, N.W. Washington, D.C. 20006.

Parker, David G., and Sam I. Thornton, June 1977, "Permeability of Fly Ash Stabilized Soils," Proceedings of ASCE Conference on Geotechnical Practice for Disposal of Solid Waste Materials, p. 68.

Reed, Gregory D., 1976, "A Proposed Procedure for Aqueous Fly Ash Evaluation," Dissertation at University of Arkansas, Fayetteville, Arkansas 72701.

Rose, Jerry G., Jeffrey A. Lowe and R. Kevin Floyd, February 1979, "Composition and Properties of Kentucky Power Plant Ash," Proceedings Fifth International Ash Utilization Symposium, NTIS, Department of Commerce, METC/SP-79/10 (Pt. 1) p. 220.

APPENDIX A

REGISTRATION LIST

Name	Address	Company
Alyassin, Zaynab	1390 S. Winchester San Jose, CA 95128	CEDEC
Anderson, Bob	8341 Sangre De Cristo Suite 9 Littleton, CO 80123	Vibroflotation Foundation Corp.
Armstrong, Joseph	2701 Prospect Ave. Helena, MT 59620	Montana Highway Dept.
Barrett, Bob	2110 Desert Hills Rd. Grand Jct., CO 81503	Colorado Highway Dept.
Belknap, Ed	3035 So. Harrison St. Denver, CO 80210	Colorado Highway Dept.
Bennett, Warren	806 Don Cubero Santa Fe, NM 87501	New Mexico Highway Dept.
Benson, Richard	3333 NW 21st Street Miami, FL 33142	Technos, Inc.
Bingham, Dave	3713 Lancelot Court Raleigh, NC 27604	North Carolina Dept. of Transp.
Bolling, Doyt	555 Zang Lakewood, CO 80225	FHWA
Bong, Larry	2911 S. Clarkson Englewood, CO 80110	Colorado Highway Dept.
Bonham, L.D.	4856 Park Avenue Bethesda, MD 20816	US Geological Survey
Brazie, Mike E.	4765 Independence St. Wheatridge, CO 80033	F.M. Fox & Assoc., Inc.
Bukovansky, Michal	3005 Quail Lakewood, CO 80215	Geo-Hydro Consulting, Inc.
Bump, Vernon	Depart. of Transp. Pierre, SD 57501	South Dakota Depart. of Transp.
Chassie, Ronald	19411 Wilderness Dr. West Linn, OR 97068	FHWA

Name	Address	Company
Church, Ed	925 E. 17th Ave. Denver, CO 80218	E. O. Church, Inc.
Cochran, Dale	1530 Dover, No. 6 Lakewood, CO 80215	Consulting Engineering Geologist
Coffee, John	P.O. Box 29 Olympia, WA 98501	FHWA
Coffin, James	P.O. Box 1708 Cheyenne, WY 82002-0199	Wyoming Highway Dept.
Collins, John	9120 Stillwater Circle Sandy, UT 84092	Hilfiker Retaining Walls
Davenport, Clark	1840 Mt. Zion Dr. Golden, CO 80401	Davenport/Hadley Limited
Dutro, Howard	16027 W. 5th Avenue Golden, CO 80401	Terrametrics, Inc.
Ealy, Carl	400 SW 7th St. Washington, DC 20590	FHWA
Eastwood, Alan	1250 So. Clermont St. Bldg. 3, Apt. 202 Denver, CO 80222	CTL/Thompson, Inc.
Enyart, Jeffrey	3333 Quebec, Suite 4020 Denver, CO 80207	The Reinforced Earth Company
Eshelman, Clarence	9002 SE McLoughlin Milwaukie, OR 97222	Oregon Depart. of Transp.
Everitt, Martin	11177 W. 8th Ave. Lakewood, CO 80225	USDA Forest Service
Foster, LeRoy	Depart. of Transp. Pierre, SD 57501	South Dakota Depart. of Highways
Franceski, John	8000 E. Girard Ave. Suite 317 Denver, CO 80231	Schnabel Foundation Company
Fredericksen, Walt	R.R. #1, Box 144K Thayer, KS 66776	Kansas Dept. of Transportation
Fretwell, William	2861 Marietta St., S.E. Salem, OR 97302	Oregon Highway Department

Name	Address	Company
Gates, Richard	224 W. 8th Avenue Vancouver British Columbia, Canada V5Y 1N5	Golder & Assoc.
Gilmore, Brandon	2359 So. Milwaukee Denver, CO 80210	Colorado Depart. of Highways
Hager, Mike	3300 Barbell Cheyenne, WY 82004	Wyoming Highway Department
Hammell, Jim	Department of Transp. Pierre, SD 57501	South Dakota Depart. of Trans.
Hanna, Bruce	3333 Quebec, Suite 4020 Denver, CO 80207	The Reinforced Earth Company
Hartman, Ray	4891 Independence, Suite 140 Wheatridge, CO 80033	E.I. DuPont, Inc.
Hilfiker, William	P.O. Drawer L Eureka, CA 95501	Hilfiker Retain- ing Walls
Himmelreich, John	1000 W. Fillmore St. Colorado Spgs., CO 80907	Lincoln-DeVore
Holmquist, Darrel	5 Columbine Lane Littleton, CO 80123	CTL/Thompson, Inc.
Hood, John	505 NW 76 Street Vancouver, WA 98665	Con-Roc International
Hood, Robert	1101 Osage Ave. Santa Fe, NM 87501	New Mexico State Highway Dept.
Huggins, Eve	1175 Emerson 305 Denver, CO 80218	Colorado Dept. of Highways
Hynes, Jeff	1313 Sherman Denver, CO 80201	Colorado Geo. Survey
Ivey, John B.	2425 So. Colo. Blvd. Denver, CO 80222	Amuedo & Ivey, Inc.
Janik, Chuck	1118 State Harrisburg, PA 17120	Pennsylvania Depart. of High- ways

Name	Address	Company
Johnsen, Ed	450 19th NE Salem, OR 97301	Oregon Depart. of Transp.
Klinedinst, Gary	400 7th Street, SW Washington, D.C. 20590	Federal Highway Administration
Lawver, Ken	68 Sherman St. Denver, CO 80203	Office of Surface Mining
Leary, Robert	1720 Peachtree Road, NW Suite 200 Atlanta, GA 30367	Federal Highway Administration
Loetterle, Don	11177 W. 8th Ave. Lakewood, CO 80225	USDA Forest Service
Long, LaRele	440 So. Carr Lakewood, CO 80226	Diamond Drill Contracting Co.
Lossmann, Horst	59 Revere Park Nashville, TN 37205	Oman Construct. Co., Inc.
Lovell, Bill	Purdue University West Lafayette, IN 47907	Purdue Univ.
Lutman, Tim	5815 Meridian Ave. N. Puyallup, WA 98371-1399	Aardvark Drill. Sales, Ltd.
McCasland, Willard	321 Sahoma Terrace Edmond, OK 73034	Oklahoma Depart. of Transp.
McDonald, Tom	9945 W. 81st Dr. Arvada, CO 80005	Soiltest Inc.
McKean, Jim	2245 Morello Ave. Pleasant Hill, CA 94523	USDA Forest Service
McKillip, Richard	Rt. 2, Box 48 Cadiz, KY 42211	Assoc. Pile & Fitting Corp.
McLoughlin, Arthur G.	7207 High Drive Prairie Village, KS 66208	Office of Surface Mining
McNab, Nancy	Box 22508 Denver, CO 80222	CH_2M Hill

Name	Address	Company
Miller, Charles	4004 Shanandoah Pl. NE Albuquerque, NM 87111	USDA Forest Service Region 3
Mok, Carson	9001 Ottawa Place Silver Spring, MD 20910	Carson K.C. Mok, Consulting Engineer
Mok, Virginia	613 G. Street, N.W. Washington, D.C. 20001	D.C. Depart. of Transp.
Moses, Thomas	P.O. Box 1693 Anchorage, AK 99510	Alaska Depart. of Transp.
O'Neil, John Michael	4906 Chimney Drive Charleston, WV 25302	West Virginia Depart. of Highways
Paige, Russell A.	7940 W. Portland Ave. Littleton, CO 80123	Harza Eng. Co.
Pakalnis, Rimas	1281 W. Georgia Vancouver, Canada V6E 3J7	Steffen Robertson & Kirsten
Payne, David	3723 Lockwood Annan, VA 22003	FHWA - EDFD
Pell, Kynric	P.O. Box 3295 Univ. Sta. Laramie, WY 82071	University of Wyoming
Pitts, David	918 Palmer Glenwood Spgs, CO 81601	Colorado Depart. of Highways
Post, John	1872 S. Ivanhoe Denver, CO 80224	Colorado Depart. of Highways
Powell, Chris	336 S. 6th West #15 Missoula, MT 59801	US Forest Service
Prellwitz, Rodney	Drawer G Missoula, MT 59806	USFS Intermtn. Forest and Range Experiment Sta.
Riedl, Gary	302 E. 5th Ave. Cheyenne, WY 82001	Wyoming Highway Department
Roberts, Robert J.	302 Fifth Street South Moorhead, MN 56560	Robert J. Roberts & Associates, Inc., Lake Agassiz Testing Laboratories

Name	Address	Company
Robinson, Charles S.	5265 McIntyre Golden, CO 80403	Mineral Systems, Inc.
Rockers, Larry	2929 Yellowstone Dr. Lawrence, KS 66044	Kansas Depart. of Transp.
Rold, John	Rm. 715, 1313 Sherman St. Denver, CO 80203	Colorado Geo. Survey
Royster, David	2200 Charlotte Ave Nashville, TN 37203	Tennessee Dept. of Transportation
Ruckman, Al	326 33 3/4 Palisade, CO 81526	Corn Construction Co.
Schatz, Barry	96 So. Zuni Denver, CO 80223	Chen & Assoc.
Shaw, Larry	918 Avenue N Grand Prairie, TX 75050	VSL Corp.
Sherman, Bill	2780 Spruce Cheyenne, WY 82001	Wyoming Highway Department
Sieger, Bob	RR3, 104 Westfield Rd. Spearfish, SD 57783	USDA Forest Ser.
Slater, Willard	SRA Box 6206 Palmer, AK 99645	Alaska Depart. of Transp.
Smith, Mitchell	600 Briarcrest Dr. Midwest City, OK 73110	Oklahoma Depart. of Transp.
Sparrow, Bob	4720 Montgomery Lane Bethesda, MD 20014	Schnabel Foundation Company
Steele, Gray	Route 3, Box 164 Central City, IA 52214	Mobile Drilling Co., Inc.
Suedkamp, Dick	96 So. Zuni Denver, CO 80223	Chen & Assoc.
Sulzle, Warren	Department of Transp. Pierre, SD 57501	South Dakota Dept. of Transp.
Summerlee, Thomas	3015 Pennsylvania Ave Colorado Spgs., CO 80907	Summerlee & Assoc.

Name	Address	Company
Thornton, Sam	340 Eng. Bldg. University of Arkansas Fayetteville, AR 72701	University of Arkansas
Vails, Delmer	Rt. 2, Box 33C Wellston, OK 74881	Oklahoma Dept. of Transp.
Vandre, Bruce	324 Fifth Street Ogden, UT 84401	USDA Forest Service R-4
Vanikar, Suneel	1000 North Glebe Road Arlington, VA 22201-4799	FHWA
Veesaert, Marlin J.	401 North Lindbergh Blvd. St. Louis, MO 63141	General Aggregate Corp.
Versaw, Ronald	12345 W. Alameda Parkway Lakewood, CO 80228	Golder Assoc.
Wright, Earl	195 Jackson Drive Frankfort, KY 40601	Kentucky Dept. of Transp.
West, Terry R.	824 Essex Street W. Lafayette, IN 47906	Purdue Univ.
Whitlow, Burrell S.	321 Walnut Avenue P.O. Box 217 Vinton, VA 24179-217	Geotechnics, Inc.
Wyllie, Duncan	224 W 8th Ave. Vancouver, B.C. Canada V5Y 1N5	Golder Assoc.
Zanbak, Caner	426 E. Fairmont #36 Rapid City, SD 57701	South Dakota School of Mines & Technology
Zeigler, Edward J.	9580 Frederick Rd. Ellicott City, MD 21043	Rummel, Klepper, & Kahl

APPENDIX B

HOSTS AND SPONSORS

Participation of the hosts and sponsors is gratefully acknowledged.

SOCIAL EVENT HOSTS

Central Mine Equipment Company
Charles O. Riggs

Hilfiker Pipe Company
Mr. and Mrs. William K. Hilfiker

Jensen Drilling Company
Mr. & Mrs. Jerry J. Jensen

COFFEE BREAK HOSTS

Schnabel Foundation Company
John Franceski and Bob Sparrow

Reinforced Earth Company
Jeffrey I. Enyart and Bruce Hanna

Mobil Drilling Company
Gray L. Steele

DISPLAY SPONSORS

Con-Roc International
John Hood

E. I. DuPont De Nemours and Company
Raymond J. Hartman

Mobil Drilling Company
Gray L. Steele

Soiltest, Incorporated
Tom McDonald

Terrametrics, Incorporated
Howard B. Dutro

Vibrofloatation Foundation Company
Robert B. Anderson

VSL Corporation
Larry Shaw and Zaynab Alyassin

APPENDIX C

Highway Geology Symposium Proceedings

- Availability List -

First (1950) Second (1951) Third (1952) Fourth (1953) Fifth (1954) Sixth (1955) Seventh (1956) Eighth (1957) Ninth (1958) Tenth (1959)	The first 10 Symposia "Proceedings" are contained in a 3 volume set available from: W. D. Bingham State Highway Geologist N.C. Department of Transportation Raleigh, NC 27611 Phone (919) 733-6911 Cost $12.50
Eleventh (1960)	Out of print
Twelfth (1961)	Out of print
Thirteenth (1962)	Out of print
Fourteenth (1963)	Out of print
Fifteenth (1964)	Professor Thomas R. Beveridge Missouri School of Mines Rolla, MO 65401
Sixteenth (1965)	Robert C. Dean Highway Material Research Lab. Limestone Street Lexington, KY
Seventeenth (1966)	W. D. Bingham State Highway Geologist N.C. Department of Transportation Raleigh, NC 27611
Eighteenth (1967)	Professor Terry West Civil Engineering Department Purdue University West Lafayette, IN 47906
Nineteenth (1968)	Robert C. Erwin West Virginia Geological & Economic Survey P.O. Box 879 Morgantown, WV 26505 $2.50 each
Twentieth (1969)	Ross Martin Illinois Experiment Station University of Illinois Urbana, IL 61801
Twenty-First (1970)	Out of print
Twenty-Second (1971)	Oklahoma Geological Survey University of Oklahoma Norman, OK

Twenty-Third (1972)	G. S. Measors Highway Geologist Virginia Department of Highways Richmond, VA 23219
Twenty-Fourth (1973)	Out of print
Twenth-Fifth (1974)	W. D. Bingham State Highway Geologist N.C. Department of Transportation Raleigh, NC 27611
Twenty-Sixth (1975)	R. G. Charboneau Chief Geologist Idaho Department of Highways P.O. Box 7129 Boise, ID 83707
Twenty-Seventh (1976)	W. A. Wisner Florida Department of Transportation Office of Materials and Research P.O. Box 1029 Gainesville, FL 32602
Twenty-Eighth (1977)	Vernon Bump Foundation Engineer South Dakota Department of Transportation Division of Highways Pierre, SD 57501
Twenty-Ninth (1978)	Dr. Kenneth N. Weaver Maryland Geological Survey Merryman Hall John Hopkins University Baltimore, MD 21227
Thirtieth (1979) (Portland, Oregon)	To be printed
Thirty-First (1980)	L. Edwin Garner Bureau of Economic Geology University of Texas at Austin University Station, Box X Austin, TX 78712 $10.00
Thirty-Second (1981)	David L. Royster Tennessee Department of Transportation 2200 Charlotte Avenue Nashville, TN 37203 $7.00
Thirty-Third (1982)	Jeffrey L. Hynes Colorado Geological Survey 1313 Sherman St., Room 715 Denver, CO 80203