PROCEEDINGS OF THE 41st ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

AUGUST 15 - 17, 1990.
Albuquerque, New Mexico

CO-SPONSORED BY

New Mexico State Highway and Transportation Department

and

New Mexico State University
Department of Civil, Agricultural and Geological Engineering

ORGANIZING COMMITTEE

Kenneth R. White, Chairman, NMSU
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Highway Geology Symposium

HISTORY, ORGANIZATION AND FUNCTION

Established to foster a better understanding and closer cooperation between geologists and civil engineers in the highway industry, the Highway Geology Symposium (HGS) was organized and held its first meeting on February 16, 1950, in Richmond, Virginia. Since then 39 consecutive annual meetings have been held in 26 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 most part, back and forth from east to west. Following meetings in Texas and Missouri in 1963 and 1964, the Annual Symposium moved to different locations as follows:

Year	HGS Location	Year	HGS Location
1965	Lexington, KY	1966	Ames, IA
1967	Lafayette, IN	1968	Morgantown, WV
1969	Urbana, IL	1970	Lawrence, KS
1971	Norman, OK	1972	Old Point Comfort, VA
1973	Sheridan, WY	1974	Raleigh, NC
1975	Coeur d'Alene, ID	1976	Orlando, FL
1977	Rapid City, SD	1978	Annapolis, MD
1979	Portland, OR	1980	Austin, TX
1981	Gatlinburg, TN	1982	Vail, CO
1983	Stone Mountain, GA	1984	San Jose, CA
1985	Clarksville, IN	1986	Helena, MT
1987	Pittsburgh, PA	1988	Park City, UT
1989	Montgomery, AL	1990	Albuquerque, NM

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 composed of approximately 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 contribution to the symposium. The officers include a chairman, vice chairman, secretary, and treasurer, all of whom are elected for a two-year term. Officers except for the treasurer may only succeed themselves for one additional term.

A number of three-member standing committees conduct the affairs of the organization. The lack of rigid requirements, routing, and the relatively relaxed overall functioning of the organization is what attracts many of the participants.

Meeting sites are chosen two or 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 protem of the Steering Committee.

The symposia are generally for two and one-half days, with a day-and-a-half for technical papers and a full-day for the field trip. The symposium usually begins on Wednesday morning. The field trip is usually Thursday, followed by the annual banquet that evening. The final technical session generally ends by noon on Friday.

The field trip is the focus of the meeting. In most cases, the trips cover approximately from 150 to 200 miles, provide for six to eight scheduled stops, and require about eight hours. Occasionally, cultural stops are scheduled around geological and geotechnical points of interest. To cite a few examples, in Wyoming, 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 the Tennessee trip provided stops at several repaired landslides in Appalachia. The Colorado field trip consisted of stops at geological and geotechnical problem areas along Interstate 70 in Vail Pass and Glenwood Canyon, while the Georgia trip in 1983 concentrated on highway design and construction problems in the Atlanta urban environment. The 1984 field trip had stops in the San Fransisco Bay area which illustrated the planning, construction and maintenance of transportation systems. In 1985, the one day trip illustrated new highway construction procedures in the greater Louisville area. The 1986 field trip was through the Rockies of recent interstate construction in the Boulder Batholith. The trip highlight was a stop at the Berkeley Pit in Butte, Montana, an open pit copper mine.

At the technical sessions, case histories and state-of-the-art papers are most common with highly theoretical papers the exception. The papers presented at the technical sessions are published in the annual proceedings. Some of the more recent proceedings may be obtained from the Treasurer of the Symposium.

Las Cruces, New Mexico
August 1990

Organizing Committee
41st Annual Highway Geology Symposium

Highway Geology Symposium

MEDALLION AWARD 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
Henry Mathis	1982
David Royster	1982
Terry West	1983
Dave Bingham	1984
Vernon Bump	1986
C. W. (Bill) Lovell	1989

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 to individuals who have made significant contributions to the Highway Geology Symposium over a period of years. The award, a 3.5" medallion mounted on a walnut shield and appropriately inscribed, is presented during the banquet at the Annual Symposium.

Highway Geology Symposium

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Highway Geology Symposium

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Mr. Willard McCasland - Chairman 1991
Materials Division
Oklahoma Department of Transportation
200 N.E. 21st Street
Oklahoma City, OK 73105
(405) 521-2677

Mr. Harry Moore - Vice Chairman 1991
Geological Engineering Supervisor I
Tennessee Department of Transportation
Geotechnical Section, P. O. Box 58
Knoxville, TN 37901
(615) 594-6219 or (615) 933-6776

Mr. Sam I. Thornton - Secretary 1990
University of Arkansas
Department of Civil Engineering
Fayetteville, AR 72701
(501) 575-6024

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State Highway Geologist
Department of Transportation
Division of Highways
Raleigh, NC 27611
(919) 733-6911

Highway Geology Symposium

STEERING COMMITTEE MEMBERS

	Term Expires
Mr. Vernon Bump Foundation Engineer Division of Engineering Department of Transportation Pierre, South Dakota 57501 Phone - (605) 773-3401	1990
Mr. John B. Gilmore Colorado Highway Department 4340 East Louisiana Denver, Colorado 80222 Phone - (303) 757-9275	1992
Mr. Joseph A. Gutierrez Mgr. Mine Planning and Development Mideast Division Vulcan Materials Company P. O. Box 4195 Winston-Salem, North Carolina 27105 Phone - (919) 767-4600	1991
Mr. Jeffery L. Hynes Colorado Geological Survey 1313 Sherman Street, Room 715 Denver, Colorado 80203 Phone (303) 866-3520	1991
Mr. Charles T. Janick Soils Engineer Pennsylvania Department of Transportation 1118 State Street Harrisburg, Pennsylvania 17120	1991
Ms. Kathleen Keller 200 Vista Del Verde Hopehull, AL 36043	1991
Mr. Harry Ludowise 6308 NE 12th Ave. Vancouver, WA 98665	1992

Mr. Marvin McCauley
California Department of Transportation 1991
5900 Folsom Boulevard
Sacramento, California 95819

Mr. Henry Mathis
Manager, Geotechnical Branch 1992
Division of Materials
Kentucky Department of Highways
Frankfort, Kentucky 40622
Phone - (502) 564-2374

Mr. David Mitchell
Chief, Geotechnical Bureau 1990
Georgia Department of Transportation
15 Kennedy Drive
Forest Park, Georgia 30050
Phone - (404) 363-7546

Mr. Gary Riedl
Chief Geologist 1992
Wyoming Highway Department
P. O. Box 1708
Cheyenne, Wyoming 82002-9019
Phone - (307) 777-7450

Dr. Terry West
Associate Professor 1991
Department of Earth, Atmospheric Science
and Civil Engineering
Purdue University
West Lafayette, Indiana 47907
Phone - (317) 494-3296

Mr. W. A. Wisner
Geologist 1990
Florida Department of Transportation
Office of Materials and Research
P. O. Box 1029
Gainesville, Florida 32602
Phone - (904) 372-5304

Mr. Burrell S. Whitlow
President 1992
Geotechnics, Inc.
321 Walnut Avenue, (P.O. Box 217)
Vinton, Virginia 24179
Phone - (703) 344-4569; 344-0198

Mr. Terry Yarger
Geologist 1991
Montana Department of Highways
2701 Prospect Avenue
Helena, Montana 59620
Phone - (406) 444-6280

Highway Geology Symposium

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Highway Geology Symposium

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Assistant Director, Soil Mechanics Bureau
NYS DOT
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State Campus, Building 7
Albany, NY 12232

Sam Thornton — 1992
Department of Civil Engineering
University of Arkansas
Fayettsville, AR 72701

Dr. R. Janardhanam — 1993
Department of Civil Engineering
University of South Florida
Tampa, FL 33620-5350

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Wyoming Highway Department
Cheyenne, WY 82202-9019

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312 Michigan Avenue
Charleston, WV 25311

Highway Geology Symposium

41st ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

August 15 - 17, 1990

Albuquerque, New Mexico

Kenneth R. White	Chairman	Department of Civil, Agricultural, and Geological Engineering New Mexico State University Box 30001, Dept. 3CE Las Cruces, New Mexico 88003
Kulathu A. Aiyer	Registration Technical Program	Department of Civil, Agricultural, and Geological Engineering New Mexico State University Box 30001, Dept. 3CE Las Cruces, New Mexico 88003
Steven M. Huddleson	Field Trip Exhibits	New Mexico State Highway and Transportation Department P. O. Box 1149 Santa Fe, New Mexico 87504
Kathryn Kretz	Field Trip Exhibits	New Mexico State Highway and Transportation Department P. O. Box 1149 Santa Fe, New Mexico 87504

41st ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

PROGRAM

WEDNESDAY, 15 AUGUST, 1990

8:00 AM to 12 Noon	REGISTRATION
10:00 AM to 5:00 PM	EXHIBITOR DISPLAY

TECHNICAL SESSION I - BALLROOM
Moderator - Dr. Kenneth White, Head, CAGE Dept., NMSU

9:00 AM	Welcome	Mr. Willard McCasland, Chairman HGS Steering Committee
9:10 AM	Opening Remarks	Dr. Kenneth White, Chairman 41st Highway Geology Symposium
9:20 AM	Keynote Address	GEOLOGY OF NEW MEXICO Dr. Russell Clemons, Head Dept. of Earth Sciences, NMSU

10:00 AM to 10:30 AM **Coffee Break**

10:30 AM to 12 Noon

GEOLOGIC HAZARDS OF NEW MEXICO - William Haneberg, NM Bureau of Mines and Mineral Resources, Socorro

HIGHWAY DAMAGE RELATED TO A FAULT NEAR PIERRE, SOUTH DAKOTA - Donley Collins and Henri Swolfs, USGS, Denver

SEISMIC RECORD VERSUS GEOLOGIC RECORD IN THE SOUTHERN RIO GRANDE RIFT REGION - Jamie Barnes, Sergent, Hauskins and Beckwith, El Paso, Texas

TECHNICAL SESSION II - BALLROOM
Moderator - Mr. Steven Huddleson, NMSHTD

1:30 PM to 3:00 PM

ROCKFALL MITIGATION ALONG I-40, COCKE AND CUMBERLAND COUNTIES, TENNESSEE - Harry Moore, Tennessee DOT

SLOPE DISTRESS AND ROCK FALL INDUCED BY THE PRESENCE OF OLD UNDERGROUND EXCAVATIONS - Robert Watters, University of Nevada-Reno and Eric Rehwoldt, GeoCon, Inc.

FIELD TESTS AND EVALUATION OF ROCKNET RESTRAINING NETS - John Duffy and Duane Smith, California DOT

PERFORMANCE ANALYSIS IN ROCKFALL SIMULATION - Gordon Elliott and Kenneth Rippere, Golder Associates, Inc.

3:00 PM to 3:20 PM **Coffee Break**

3:20 PM to 5:00 PM

DESIGN OF GEOGRID WALL WITH WICK DRAINS IN TUCUMCARI, NEW MEXICO - Edward Rector and Richard Lueck, NMSHTD

GEOGRID-EXPANSIVE CLAY EMBANKMENT - Sam Thornton and Michael McGuire, University of Arkansas

CREATING AN ELEVATED CATCHMENT AREA USING A PRECAST MODULAR WALL SYSTEM - Richard Cross, NY State Thruway Authority

17 MILES TO MOUNT ST. HELENS: OPERATIONAL ASPECTS OF THE GEOTECHNICAL INVESTIGATION - George Deardorff and David Findley, Golder Associates, Inc.

THURSDAY, 16 AUGUST 1990

8:00 AM to 4:00 PM	FIELD TRIP
5:30 PM to 9:00 PM	SOCIAL HOUR-BANQUET Speaker - Dr. Lawrence Lattman, President, NMIMT, Socorro

FRIDAY, 17 AUGUST 1990

8:00 AM to 12 Noon EXHIBITOR DISPLAY

TECHNICAL SESSION III - BALLROOM

Moderator - Dr. Joseph Finney, CAGE Dept., NMSU

8:30 AM to 10:00 AM

ANALYSIS AND DESIGN OF TIEBACK WALL NO. 5 IN STEUBENVILLE, OHIO - Richard Humphries and Gordon Elliott, Golder Associates, Inc.; Gerald Cafarelli and John Hollenbaugh, URS Consultants; Eugene Geiger, OH DOT

ALTERNATIVE METHODS FOR RETAINING WALLS - Peter Nicholson and Spark Johnston, Nicholson Construction Co.

T-WALL-ENGINEERED FOR ECONOMY - Thomas Neel, The Neel Co.

10:00 AM to 10:20 AM **Coffee Break**

10:20 AM to 12 Noon

SLOPE FAILURE PROBABILITY FOR LAYERED SOILS - Sam Thornton and Steven Garret, University of Arkansas

FEDERAL HIGHWAY ADMINISTRATION'S TECHNOLOGY TRANSFER ACTIVITIES IN GEOTECHNICAL ENGINEERING - Chien Tan Chang, FHWA

DATA ACQUISITION SYSTEM FOR MECHANICAL DUTCH CONE PENETROMETER - Steven Huddleson, NMSHTD

FLORIDA'S MINERAL AGGREGATE CONTROL PROGRAM - William Wisner, Florida DOT

12 Noon Symposium Adjourns

PAPERS

GEOLOGY OF NEW MEXICO

Russell E. Clemons
New Mexico State University, Las Cruces, NM 88003

Introduction

This paper is intended as a geological overview of New Mexico. The state's enchanting landscape and its natural resources are like a huge museum showcase. Layers of rocks exposed in highway cuts, valley walls, and mountain slopes represent pages of earth history that geologists "read" and interpret to learn how New Mexico has appeared for the past 2 billion years. Figure 1 contains an abbreviated version of this historical record. Landforms such as canyons and mountain ranges determined routing of early trains and roads in the state. Subsequently, discovery of mineral deposits in the mountains, oil and gas in the northwest and southeast corners of the state, and coal in San Juan and Colfax Counties provided purposes of routing and building other highways and railroads throughout the state.

New Mexico contains parts of five physiographic provinces: Colorado Plateau, Rocky Mountain, Great Plains, Basin and Range, and Datil-Mogollon (Fig. 2). Each province contains geologic materials and features, the combination of which, distinguish it from the other provinces by a distinct pattern of landforms related to geologic features that evolved through time. The Colorado Plateau is a region of scarped tablelands with broad valleys and local canyons cut in Mesozoic and Cenozoic rocks. Volcanic features, like Mount Taylor, and young basalt flows are widespread along the southern and southeastern borders. The Rocky Mountains include the high mountain areas and Rio Grande gorge of north-central New Mexico. Much of this province is composed of Tertiary and Precambrian rocks (Fig. 3). The Great Plains consists of high plateaus cut by Canadian and Cimarron River canyons in the northeast. Extensive basalt flows and Miocene alluvium (Ogalla Formation) discontinuously cover older rocks. Southward the province becomes broad lowland plains and terraced valleys of the Pecos and Canadian Rivers. Dissolution of Permian limestone and gypsum has produced extensive karst topography in the Santa Rosa-Vaughn-Fort Sumner region. Farther south an extensive cover of eolian and fluvial sediments mantle the Ogallala caprock. The Basin and Range province is characterized by fault-block mountain ranges separated by broad, elongate basins (valleys) having internal drainage. Narrow Rio Grande and Gila River valleys are incised into basin fill and older rocks. The Datil-Mogollon province, a transition area between the Basin and Range, and Colorado Plateau, is a volcanic terrane dominated by high tablelands, broad structural basins, and scattered fault-block ranges.

Precambrian Era
(4,600 to 570 million years ago)

Precambrian terrane underlying New Mexico is composed of granitic plutons intruded into regionally metamorphosed rocks including schists, gneiss, quartzite and marble. Precambrian rocks are exposed in cores of the Sangre de Cristo, Tusas, Zuni, and Big Burro Mountains, Pedernal Hills, and along the bases of uplifted fault-block ranges like the Nacimiento, Sandia, Manzano, Oscura, San Andres, Organ and Caballo Mountains. Highway cuts in Tijeras Canyon and the Sangre de Cristo (Fig. 4), Tusas, Big Burro, and Organ Mountains provide good exposures of some of these rocks. Subsurface data obtained from hundreds of exploration wells drilled into Precambrian basement rocks provide information making possible interpretations between the widely scattered, limited exposures (Fig. 3).

About 2 billion years (b.y.) ago, shallow seas covered parts of the state and extensive beds of sand and mud were deposited. Locally, algal colonies formed mats and small mounds along the coastlines. Contemporaneous volcanic activity is recorded by the interbedded basalt flows and rhyolitic tuffs and flows. The lands were barren and devoid of plant and animal life so weathering and rapid erosion provided abundant sediments to the ocean floor. Then the seas withdrew and upheaval of lofty mountains was concomitant with metamorphism of most of the sedimentary and volcanic rocks, additional volcanic activity, and emplacement of huge masses of granite, granodiorite and syenite. Evidence exists in many areas for at least two major periods of Precambrian orogenesis (mountain building). Radiometric ages range from 1.9 b.y. on some rocks in northern New Mexico to as young as 1.0 b.y. on Precambrian rocks in the south.

Several hundred million years (m.y.) passed as the seas withdrew and erosion wore down the mountains to an extensive, gently undulating surface. Apparently no major mountain building occurred at this time because there is little discordance between the younger Precambrian rocks and overlying lower Paleozoic rocks.

Lower Paleozoic
(570 to 360 m.y.)

Outcrops of Lower Paleozoic rocks are sparse and limited to lower slopes of uplifted fault-blocks in the southern part of the state. Syenite and granite form a major part of the Florida Mountains, near Deming. Radiometric ages of 523 and 503 m.y. indicate these rocks were emplaced during Late Cambrian or Early Ordovician times. Small exposures of similar Cambrian-Ordovician, alkali-rich, igneous rocks have been reported from around New Mexico. These shallow plutons were unroofed in southern New Mexico, by erosion of overlying rocks, by Early Ordovician time.

Southern New Mexico was a barren gently rolling, lifeless plain as the Paleozoic (ancient life) Era dawned. Northern New Mexico was probably more hilly and mountains. Upper Cambrian and Lower Ordovician Bliss Sandstone and El Paso Formation were deposited near the shoreline of a shallow sea that encroached the land from south or the southwest. The Bliss Sandstone, varying in thickness from 100 to 150 ft., represents a beach and shallow-marine sand containing scattered fossil fragments. The Ignacio Formation in San Juan County represents a beach sand deposited along an eastward transgressing shoreline during Late Cambrian. As Early Ordovician sea transgressed northward, about 1,000 ft. of limy muds were deposited in intertidal and shallow subtidal flats in the southern and northeastern parts of the state. The region may have resembled the present-day Bahama Banks--but without any vegetation. These layers of sediments were then cemented and compacted during burial by overlying sediments to form the limestones, dolostones, and chert that make up the El Paso Formation (Arbuckle Group in northeast). Fossils found in these rocks indicate the seas teemed with trilobites, brachiopods, snails, echinoderms, sponges, cephalopods and several types of algae.

Following deposition of the El Paso Formation, the sea withdrew, and the state was briefly (for about 20 m.y.) land again. Shallow seas returned during Late Ordovician, before extensive erosion took place, and about 400 ft of Montoya Dolostone were deposited over much of southern New Mexico. Parts of northwestern New Mexico may have been islands during the Ordovician, but most of the state was probably covered by a shallow sea. Later erosion removed Ordovician rocks from central and northern New Mexico.

The Fusselman Dolomite was deposited during Middle to Late Silurian time as the sea again transgressed the state. This dolostone remains only in the southern and southeastern parts of the state, thinning northward from 1400 feet south of Deming. Extent of Middle Silurian seas is unknown, but most of central and northern New Mexico was undergoing erosion during Late Silurian time.

During Middle and Late Devonian time a blanket of silt and mud (now called the Onate, Sly Gap, and Contadero Formations and Percha Shale) was laid down on a shallow sea floor and in local stagnant basins or lagoons. These fine-grained sediments were washed off coastal plains and low, deeply weathered lands of central and northern New Mexico. Fish, brachiopods, bryozoans, and corals were common dwellers of the shallow seas. Primitive amphibians may have inhabited swamps and lowlands forested with tree ferns, horsetail rushes, and lycopods (spike-leafed trees) to the north. Upper Devonian rocks under the San Juan basin include a few hundred feet of dolostone, limestone, sandstone, and shale belonging to the Aneth, Elbert and Ouray Formations. The sea withdrew yet again and the shoreline moved south or southwestward during latest Devonian and dawning "hours" of Mississippian times.

Upper Paleozoic
(360 to 245 m.y.)

Mississippian rocks probably were deposited over most of New Mexico. Subsequent erosion removed much of the rocks in northern New Mexico. Remnants, less than 100 feet thick in most places, include the Arroyo Penasco Group and Log Springs Formation.

As the warm, shallow ocean advanced over southern New Mexico about 345 to 320 m.y. ago, up to 1000 ft. of sediments were laid down. The oldest Mississippian deposits were silty, limy muds (Caballero Formation) found in only the southern part of the state. Gradually, the shoreline moved northward and limy sediments of the Lake Valley Formation were deposited on a continental shelf covered with shallow marine water. Crinoids were so prolific that some limestone beds are made up entirely of their fragments. Fossil brachiopods, bryozoans, trilobites and snails are also abundant in these Mississippian rocks. Locally, mounds up to 400 ft. high (like the Muleshoe Mound south of Alamogordo) formed near the edge of the continental shelf. The fine-grained, thin-bedded, relatively unfossiliferous sediments composing the overlying Rancheria and Helms Formations in the south, indicate deposition in deeper water of a probably restricted basin. Following deposition of the Helms Formation the seas withdrew, this time probably as a result of the land being uplifted to the north and thus southward tilting of some of the Mississippian and older beds.

Subsequently, shallow marine waters again advanced over the slightly truncated beds and by Middle Pennsylvanian time covered much of the state. A narrow, north-trending shelf was positioned along the area of the Sacramento Mountains. West of this shelf, the Orogrande basin extended to west of Las Cruces. Limy fossiliferous sediments deposited on the shelf include up to 3,000 ft of basal sandstones, thin-bedded limestone, and algal mounds like those seen in Dry Canyon (Fig. 5) between Alamogordo and Cloudcroft. The deeper marine sediments include more dark shale and thin-bedded black limestones. These Pennsylvanian strata are interpreted as transgressive-regressive deposits formed as a result of worldwide sea-level fluctuations. During low stands of sea level the Orogrande basin was the site of muddy, highly saline waters in which some gypsum beds were precipitated. Pennsylvanian rocks in the Organ and San Andres Mountains have been subdivided for mapping purposes into the La Tuna, Berino, Lead Camp, and Panther Seep Formations. These are approximately correlative with the Gobbler, Beeman, and Holder Formations in the Sacramento Mountains, the Magdalena Group of central and northern New Mexico, and the Horquilla Formation of the southwestern corner of the state.

Red conglomerates, sandstones, and shales (Fig. 6) in the Sangre de Cristo, Laborcita, and Abo Formations record a time of nonmarine deposition during Early Permian. Marine Hueco Limestone interfingers with terrestrial Abo in the southern Sacramento Mountains and in the Robledo Mountains, west of Las Cruces. Animal tracks, mudcracks, and fossil plants found in the Abo indicate a terrestrial origin, probably deltaic floodplains between mountains to the northeast and open seas to the south. The Sangre de Cristo Formation, southeast of Santa Fe, ranges in thickness from 700 to 5300 ft and records the uplift of ancestral Rocky Mountains. Gradationally overlying the Abo and Sangre de Cristo redbeds are up to 4,000 ft of red, brown, and yellow mudstone, gypsum, and limestone (Yeso Formation) recording deposition on an arid to semiarid coastal plain and shallow seafloor as the ocean returned again.

A sheet of white quartz sand filled the late Yeso seas; the resulting Glorieta Sandstone, about 200 ft thick, prominently caps Glorieta Mesa (Fig. 7). Its cliffs are a familiar sight to travelers on the Santa Fe Railway at Glorieta Pass. Broad seas then spread over all but northern New Mexico and a thick (600 to 1000 ft) San Andres Limestone, was laid down. Much oil is produced in southeastern New Mexico from this unit of limestones and dolostones. The rich agricultural region stretching from Roswell to Artesia depends on underground water gained from the San Andres Limestone; rain and snow falling on the Sacramento Mountains, seeps underground into the cracks and caverns within the San Andres, and flows eastward downslope to the Pecos Valley.

About 5,000 ft of Upper Permian marine rocks near Carlsbad, indicate the southeast corner of the state remained part of an ocean basin, called the Delaware basin, until the end of Permian time. While the Yeso Formation was being deposited to the north and northwest, the Delaware basin was rimmed by a low, broad bank of Victorio Peak Limestone. In the basin, to the south, black sandy limestone and black shale of the Bone Spring Formation were deposited. The deep Delaware basin was rimmed by magnificent towering barrier reefs, the Goat Seep and Capitan reefs that now are host to Carlsbad Caverns. These reefs were similar to the present-day Great Barrier Reef of Australia. The Capitan reef is about 400 miles long and other than oceanward channels cut through to the south, completely encircled the 10,000 square-mile Delaware basin. Its northwestern border is now marked by the southwest-trending front of the Guadalupe Mountains southwest of Carlsbad; its north edge was east-northeast of Carlsbad, and it extended southward into West Texas. During latest Permian time, the Ochoan Series filled up the Delaware basin as about 5,000 ft of evaporite (gypsum, salt, potash) beds were deposited. These include, in ascending order, the Castile Gypsum, Salado Formation, and Rustler Formation. The WIPP site is in the Salado Formation, composed largely of salt.

Mesozoic
(245 to 66.4 m.y.)

The Mesozoic (middle + life) Era is divided into Triassic, Jurassic, and Cretaceous Periods. During Triassic time, southern New Mexico was probably low hills and extensive plains basking under a hot, semiarid climate. Erosion may have provided varicolored sediments now seen to the north in the Santa Rosa-Tucumcari areas. During this time, sands and muds eroded from New Mexico were carried westward to northeastern Arizona where they now form the Moenkopi Formation, the brilliant reds and purples of the Painted Desert region. Uplands arose in late Triassic time in southwestern New Mexico. Along with mountains in south-central Colorado, these highlands were torn apart by water and wind, and the detritus was swept into sheets of brightly colored sand and shale. These beds are thickest (about 2,000 ft) along the New Mexico-Texas line east of Roswell and in west-central New Mexico (near Grants) extending westward into northeastern Arizona. The eastern Triassic rocks are the redbeds of the Dockum Group with the lower Santa Rosa Sandstone and the upper Chinle beds. The northwestern rocks are the Chinle Formation overlain by the redbeds of the Wingate Sandstone.

The Chinle Formation is of special scenic interest as its beds contain the silicified trees so well shown at Petrified Forest National Monument. These varicolored rocks--red, purple, green, and gray--also decorate the Painted Desert area, the wide valley of Rio San Jose east of Laguna, and flank I-40 from Texas westward almost to Clines Corners. Some beds are weathered "ash" beds from volcanoes in the highlands. In contrast to the underlying marine Paleozoic rocks, these Triassic beds were deposited on land by streams and in shallow lakes. Beasts that roamed New Mexico were amphibians--such as the thick-skulled stegocephalians--and reptiles of the crocodile-like clan, the phytosaurs.

New Mexico was featureless rolling prairie, with scattered low hills in the northwest, during most of Jurassic time. In the late part of the period, the Sundance-Curtis sea and its shoreline lagoons spread down from the north into northwestern New Mexico. Sand dunes on its southeastern shores consolidated into the cross-bedded Entrada Sandstone; its reddish brown cliffs rim the Rio San Jose Valley near Grants and Gallup. Gypsum and limestone of the Todilto Formation were precipitated, in an extensive salt-water lake that covered most of northwestern and north-central New Mexico. This gypsum is the bed mined between Albuquerque and Santa Fe, and near San Ysidro on White Mesa. The Todilto is overlain by reddish sands and silts of the Summerville Formation, washed chiefly from the south, and by the multicolored sands of the Zuni Sandstone (exposed at El Morro). Uppermost Jurassic rocks are the stream and wind-blown sands and clays of the varicolored Bell Ranch and Morrison Formations laid down in northern New Mexico.

Petrified wood and bone fragments are abundant in the Morrison beds--along with uranium--but no fossil finds in New Mexico equal those in the Morrison Formation of Dinosaur National Park, Utah.

The Cretaceous Period was one of great contrast in New Mexico. During Early Cretaceous time, most of the central and northern parts of the state were lowlands torn by erosion, while thick piles of conglomerate, sandstone, and shale accumulated in the Pedregosa basin in the southwestern corner. About 10,500 ft of Lower Cretaceous terrestrial and marine shale, sandstone, and limestone are well exposed in the Peloncillo, Animas, Big Hatchet, Little Hatchet and East Potrillo Mountains. In ascending order, these rocks comprise the Hell-to-Finish, U-Bar, and Mojado Formations. They were deposited as alluvial fan, fluvial, lacustrine, and interbedded shallow marine sediments along the northern side of a west-northwest-trending basin.

Thousands of feet of marine and terrestrial sediments were laid down in northern and central New Mexico during Late Cretaceous time, whereas most of the southern part of the state was above sea level and was being eroded. The shorelines made parallel northwest-trending bands across the state. These are now marked by beach sands. Northwestern and central New Mexico was a battleground of the land and sea, with the beaches advancing and retreating fifty or a hundred miles during an instant of geologic time. Stream sands and coal beds lie landward from the beach sands which, in turn, mingle seaward with black limy shales that were flushed into the seas. The lowest of these rocks is the Dakota Sandstone. Above is the black Mancos Shale, which in turn is overlain by the Mesaverde Group.

Transitions from coal swamps and stream sands to beach deposits and then into marine black shales is characteristic of the Mesaverde in northwestern and north-central New Mexico. To the northeast, beds of the same age were laid down in an extensive muddy sea that stretched far to the east; the Pierre Shale and Niobrara chalky limestone that underlie the plains northeast of Las Vegas are typical. The cliff-forming sandstones, and coal beds near Gallup are part of the Mesaverde Group and rim the entire San Juan basin; coal at Madrid, southeast of Socorro, and east of Carrizozo, also is in the Mesaverde. Above are similar rocks such as the Kirtland Shale, Pictured Cliffs Sandstone, and Fruitland Formation that underlie valleys cut in the shales and cliffs carved from the sandstones in the northwest corner of New Mexico at Bisti and near Farmington.

Toward the end of the Cretaceous, the Laramide orogeny began, and New Mexico along with most of North America emerged from beneath the seas, to be high and dry to the present. The orogeny, an extensive upheaval of the earth's crust, saw uplift of the San Juan Mountains area in southwestern Colorado with large volcanoes erupting nearby. Fragments of the eroded mountains and debris from these andesitic volcanoes were flushed southward by streams and steam to settle as thick piles of mud, sandstone, and conglomerate, the Animas Formation in the San Juan Basin.

Similarly, mountains arose during Late Cretaceous time in north-central New Mexico and south-central Colorado, about on the site of the present-day Sangre de Cristo range northeast of Taos, and shed gravel, sand, and muds into the Raton basin. Alluvial fan gravels and sands grade eastward into dark muds and coals laid down in swamps and on floodplains. These rocks now cap mesas northwest of the Santa Fe Railroad from Raton southward--the cliff-forming Trinidad Sandstone and the dark siltstones, sandstones, black shales, and coal beds of the Vermejo (Fig. 8) and Raton Formations. Early Laramide deformation in represented in southwestern New Mexico by the Upper Cretaceous Ringbone Formation.

Fishes, amphibians, reptiles, snakes, clams, and waterbugs lived in the streams and lakes; hordes of reptiles (up to 100 ft long), and tiny primitive mammals inhabited the land; flying reptiles, the first known birds, and numerous winged insects soared and buzzed about. Interbedded marine shales and limestone contain abundant clams, oysters, corals, and ammonites (coiled cephalopods). Deciduous trees--oak, maple willow, sassafras, poplar, and birch--became numerous, along with flowering plants and more modern conifers.

The Laramide orogeny began toward the end of the Cretaceous and continued in the early Tertiary. The orogeny, a series of mountain-building events, affected the entire Rocky Mountain region from Mexico to Canada. During this time, New Mexico emerged from the seas to remain a land area to the present day. The entire state was continuously shaken by earthquakes as mountains were uplifted and millions of years of geologic records destroyed by erosion of the land area.

Cenozoic Era
(66.4 m.y. to present)

The Cenozoic (recent + life) Era is the final chapter of geologic time; it is divided into the Tertiary and Quaternary Periods. The landscape we see today in New Mexico (along with its flora and fauna) was formed by Cenozoic events. Because Cenozoic fossils and rocks top the stratigraphic column, and are at or close to the earth's surface, more information is available about this era than preceding eras.

Scenery in north-central New Mexico (Fig. 9) may have been similar to today's, with mountains in the same general areas as the present-day Sangre de Cristo, Nacimiento, San Juan, and Brazos ranges. Coarse gravels were deposited along the mountain fronts, but out in the adjoining lowlands, floodplain sands and varicolored lake-bed clays settled. Low areas were "basins" of deposition where thick masses of sediments accumulated--the Raton and Poison Canyon Formations in the Raton basin near Raton, the Animas, Nacimiento, and San Jose Formations in the San Juan basin north and northwest of Cuba, seen along NM-44, and the Galisteo Formation in the

Galisteo basin south and southwest of Santa Fe. Reddish Galisteo rocks crop out along I-25 at La Bajada Hill about 20 miles southwest of Santa Fe.

Silicified wood, chiefly of pines but with some oak and poplar, is abundant in the Galisteo Formation. Large logs, up to 6 ft in diameter and 135 ft long, have been found. In the great swamps of the Raton basin, where the climate was much like that of Georgia today, tall reeds, water lilies, fig trees, palm trees, magnolias, and sycamores grew in profusion, and contributed to the thick coal beds now mined there. The early ages of the Cenozoic saw the spectacular rise of the mammals to dominance over reptiles on land; numerous remains of the early mammals are found in the Nacimiento and San Jose formations, including the famous Puerco and Torrejon faunas.

Redbeds of the Baca Formation were laid down on the north flank of low mountains that extended from near Quemado toward Socorro. Ancient hills near present-day Sierra Blanca shed rock fragments that accumulated near Capitan as the varicolored Cub Mountain Formation. Deeply eroded uplands northwest of Elephant Butte Reservoir supplied gravels and sands that mingled with andesitic volcanic debris as the upper part of the McRae Formation in central Sierra County. The Love Ranch, Lobo, and Skunk Ranch Formations near Las Cruces, Deming, and Playas formed from debris shed from adjacent mountain ranges. Many of the weathered greenish and purplish Hidalgo volcanic rocks in southwestern New Mexico were extruded and beneath the surface magmas (molten rocks) cut into older rocks. Vapors and hydrothermal fluids from the magmas emplaced some of New Mexico's vast ore deposits during this time.

Beginning in late Eocene and continuing through the Oligocene to early Miocene time, southwestern New Mexico literally exploded. First, andesitic volcanoes erupted immense quantities of lava along with dust-size to huge-boulder-size fragments. The eruptions were probably accompanied by torrential rains that mixed with debris on the volcanic slopes to form mud flows that plunged downslope to cover adjacent lowlands. Collectively, these rocks, several thousand feet thick, make up the Palm Park Formation and Orejon Andesite which are exposed in the Dona Ana, southern Robledo, and Organ Mountains. The Espinaso, Cuchara, and Huerfano Formations of north-central and northeastern New Mexico are probably contemporary deposits. Igneous sills and dikes (Fig. 10) in the Sacramento Mountains were emplaced at this time.

A second, more violent surge of volcanic eruptions intermittently jarred the region for another 15 m.y. Most of the rocks from this period are light-colored (rhyolitic) ash-flow tuffs and flows that form the Datil-Mogollon plateau, southern Organ Mountains and Dona Ana Mountains near Las Cruces, and the bulk of most other ranges in southwestern New Mexico. Sierra Blanca near Ruidoso is a huge, volcanic mass formed at this time. Widespread volcanic activity continued into the Miocene time which began about 24 m.y. ago.

Rhyolites, pumice, and perlite in the southwest, as well as in other parts of the state, covered wide areas. Mount Taylor, towering up to 11,389 ft near Grants and visible on the western skyline from Albuquerque, is a huge volcano. Shiprock and Cabezon Peak are eroded cores of ancient volcanoes and the Taos plateau is covered with basalt flows and volcanoes of late Tertiary age. Valle Grande caldera, west of Los Alamos, is in a late Tertiary-Quaternary volcanic mass. Bandelier National Monument is within an ash-flow tuff (Fig. 11) erupted from the caldera about 1 m.y. ago. Capulin Mountain, east of Raton, is a huge recent cinder cone and is surrounded by numerous basaltic lava flows that cap the High Plains from Raton eastward to Clayton. The very fresh black basalt flows near Carrizozo and in the valley of Rio San Jose near Grants may be less than 1,000 years old. Numerous mesas along the Rio Grande Valley from the Colorado line to El Paso (Fig. 3) are capped by black basalt flows of late Tertiary and Quaternary ages.

Most of the present-day mountains were uplifted in early Miocene time, following climax of the great volcanic eruptions. This uplifting took place along one side of huge mountain masses, forming tilted fault blocks like the Sandia, Manzano, San Andres, Caballo, Guadalupe, and Sacramento mountains. Strata in the Sandia Mountains, for example, dip to the east, but were uplifted as much as four miles along a west-bounding fault zone. The uplifting took place slowly, and is continuing today as the Albuquerque area, along with the Rio Grande Valley southward to Socorro, is one of the most active earthquake areas in the state. Concurrent with uplift, other blocks sank, forming graben basins which were filled with rock debris from the adjoining uplifts. A series of irregular grabens, now followed by the Rio Grande, cut north-south across the state. Within this complex graben, and around the bordering ranges, sandstones and siltstones of the Santa Fe Group (Fig. 12) were deposited--these red, yellow, orange, and cream rocks are eroded in many places, such as near Santa Fe, to "badlands" characteristic of the landscapes along the Rio Grande Valley from Espanola southward to El Paso. Freight-car loads of mammal fossils have been shipped to museums from outcrops near Espanola. Similar sands and gravels of the Gila Conglomerate filled basins amid the mountains of southwestern New Mexico. East of the mountains of central New Mexico gravels of the Ogallala Formation were spread onto the western edges of the High Plains. They now cap the plains as well as make picturesque bluffs east of the Pecos River and southeast of Tucumcari--the "caprock" of those areas.

The final phases of landscape development occurred during the Pleistocene, the recent Ice Ages, about 2 m.y. to 10,000 years ago. Valley glaciers occupied some of the higher parts of the state, as far southward as Sierra Blanca; large lakes filled many of the closed basins, such as those near Estancia, Playas, Animas, Alamogordo, and west of Lordsburg. The glistening white gypsum dunes at White Sands National

Monument, built into 50-ft-high mounds leeward of Lake Lucero, formed during this time.

The Rio Grande, in its present valley, probably is only as old as mid-Pleistocene, born during late uplift of its headwater mountains, the San Juan and Sangre de Cristo ranges in southern Colorado and northern New Mexico--initiated by floods of meltwaters from waning mountain glaciers. Some of the lower terraces (benches) along the Rio Grande and its tributaries formed about 2,000 years ago.

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Era	System	Southwest	Southeast	Northeast	Northwest
CENOZOIC		Gila Cgl, Santa Fe Gp, volcanic rocks	Ogalla Fm, Cub Mountain Fm, Sierra Blanca volcanic rocks	Ogallala Fm, volcanic rocks, Poison Canyon Fm, Raton Fm	Santa Fe Gp, Los Pinos, San Jose, Nacimiento Fms; Ojo Alamo Ss, volcanic rocks
MESOZOIC	K	Ringbone, McRae, Mojado, U-Bar, Hell-to-Finish Fms	Mesaverde Gp; Dakota, Mancos, Tucumcari, Mesa Rica Fms	Vermejo, Niobrara Fms; Trinidad, Dakota Ss; Pierre, Graneros Sh	Kirkland Sh, Fruitland Fm, Pictured Cliffs, Ss, Lewis Sh, Mesaverde Gp, Mancos Sh, Dakota Ss
	J		Exeter, Morrison Fms	Exeter, Morrison Fms	Morrison, Todilto Fms; Entrada Ss
	Tr		Chinle Fm, Santa Rosa Ss	Chinle Fm, Santa Rosa Ss	Chinle Fm
PALEOZOIC	Pr	Concha Ls, Epitaph Dol, Colina Ls, Earp Fm	Salado, Castile Fms; Artesia Gp; San Andres, Yeso, Abo-Hueco Fms	San Andres, Yeso, Abo Fms; Glorieta Ss	San Andres, Yeso, Abo Fms; Glorieta Ss
	Pn	Horquilla Ls	Magdalena Gp; Gobbler, Beeman, Holder Fms	Sangre de Cristo Fm, Magdalena Gp	Hermosa Gp
	M	Paradise Fm, Escabrosa Ls	Caballero, Lake Valley, Rancheria, Helms Fms		Leadville Ls, Arroyo Penasco Gp
	D	Percha Sh	Sly Gap, Percha, Onate Fms		
	S		Fusselman Dol		
	O	Montoya, El Paso Fms	Montoya, El Paso Fms		Aneth, Elbert, Ouray Fms
	C	Bliss Fm	Bliss Fm		Ignacio Ss
	pC	gneiss, granite	gneiss, granite	gneiss, granite	gneiss, granite

Figure 1. Stratigraphic chart containing the names of major rock units present in the state (pC: Precambrian, C: Cambrian, O: Ordovician, S: Silurian, D: Devonian, M: Mississippian, Pn: Pennsylvanian, Pr: Permian, Tr: Triassic, J: Jurassic, K: Cretaceous).

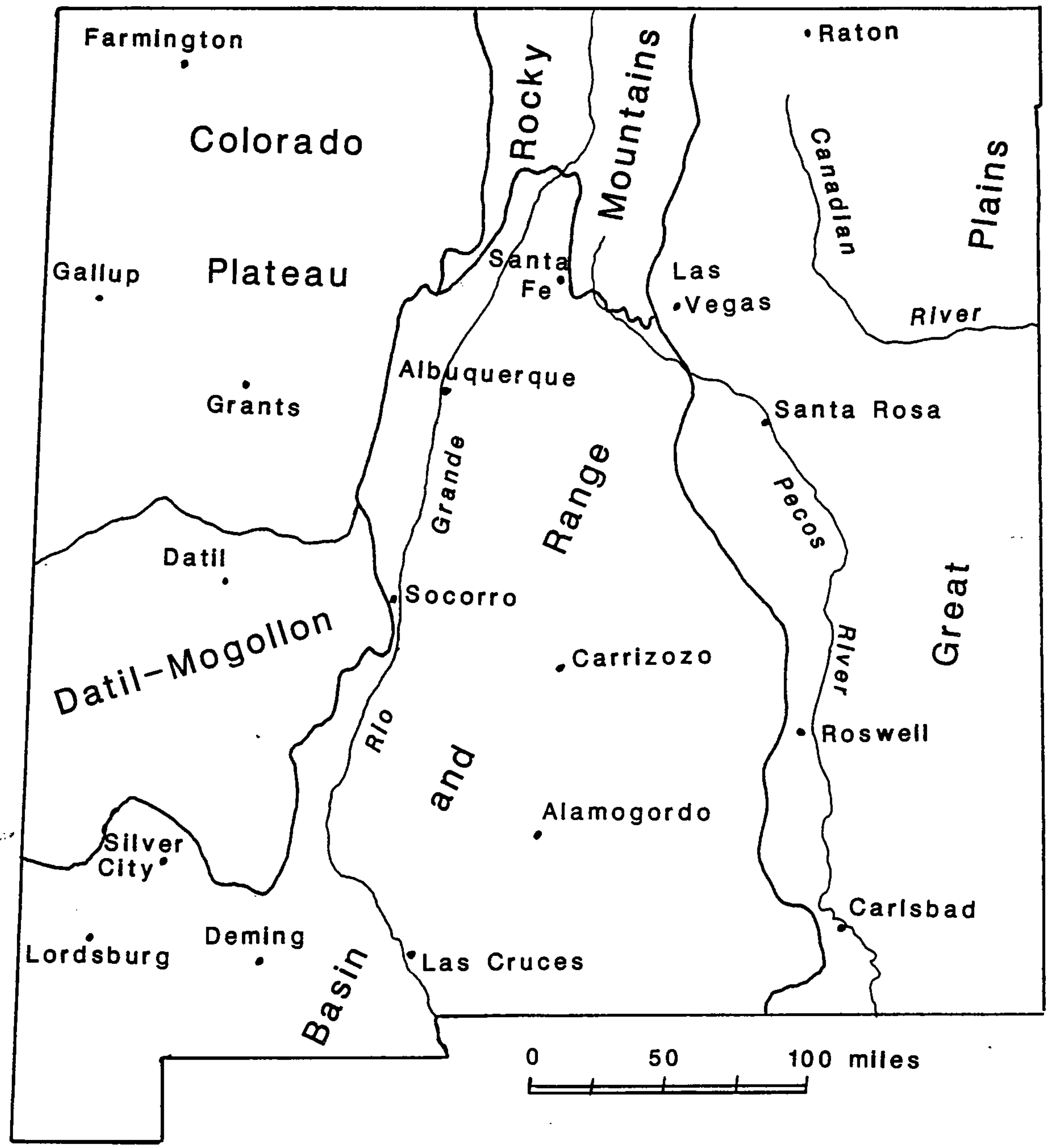


Figure 2. Locations and extent of the five physiographic provinces in New Mexico.

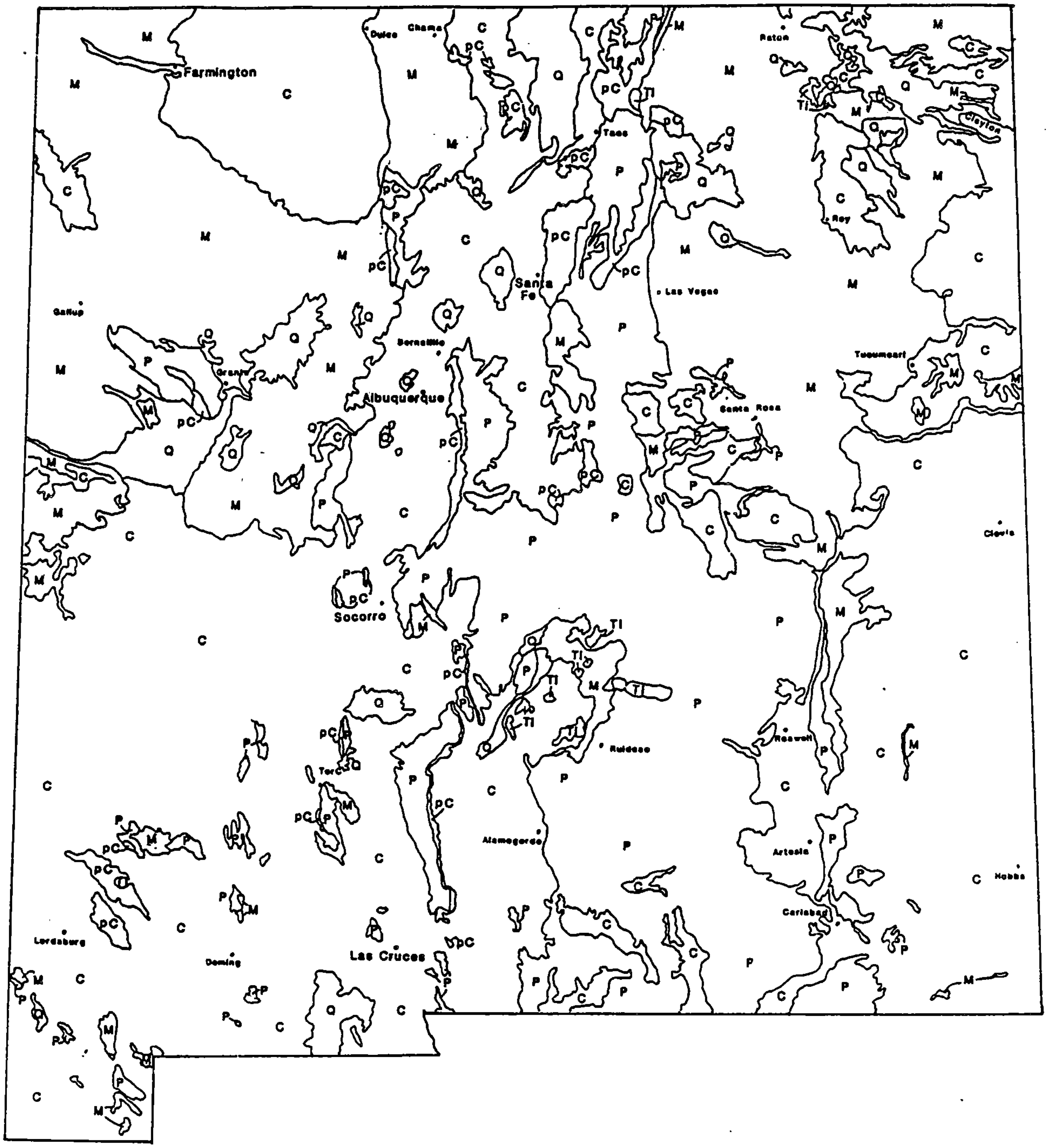


Figure 3. Generalized geologic map of New Mexico (pC: Precambrian, P: Paleozoic, M: Mesozoic, C: Cenozoic, Q: Late Tertiary and Early Quaternary basalts, Ti: Tertiary intrusive igneous rocks).

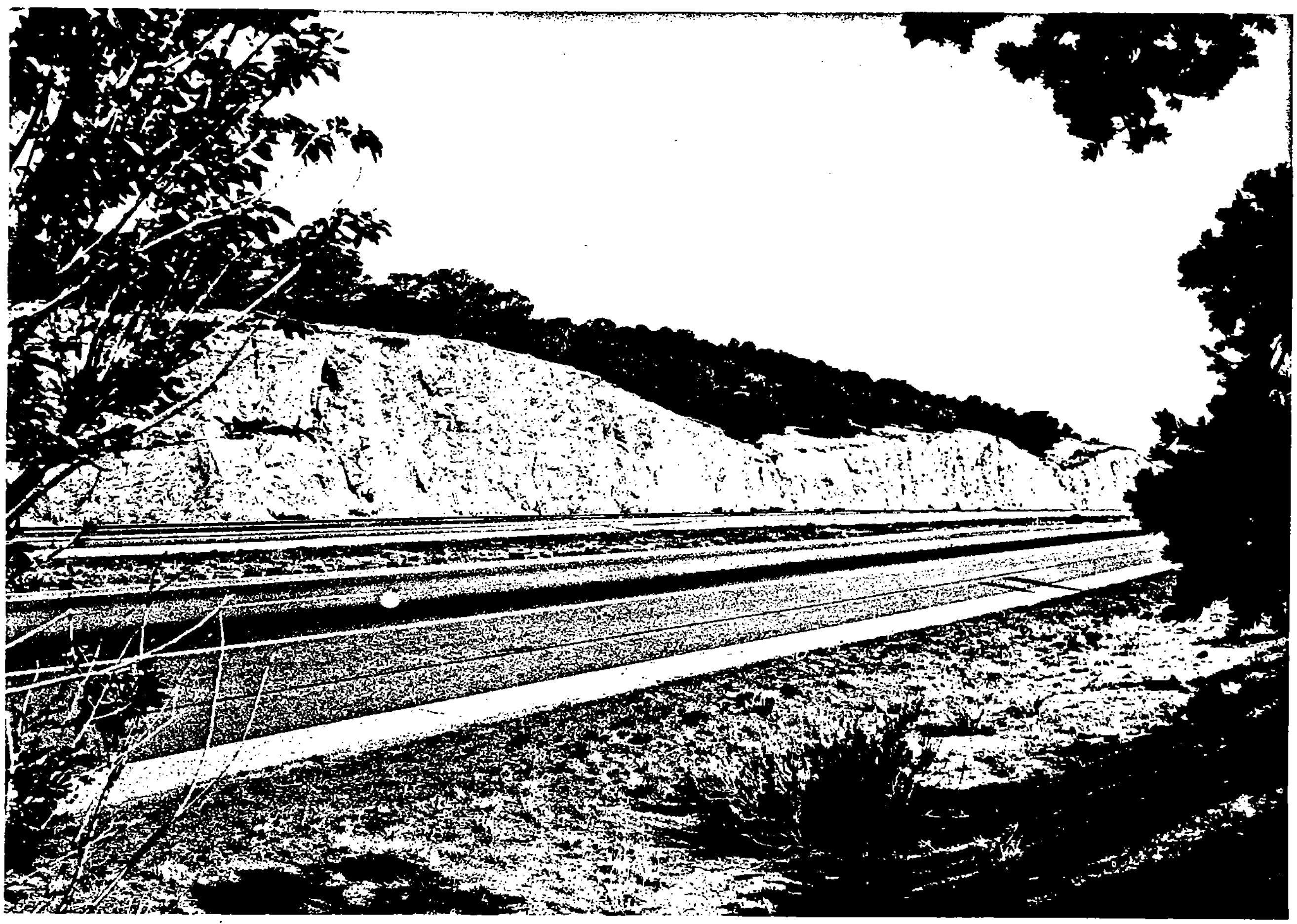
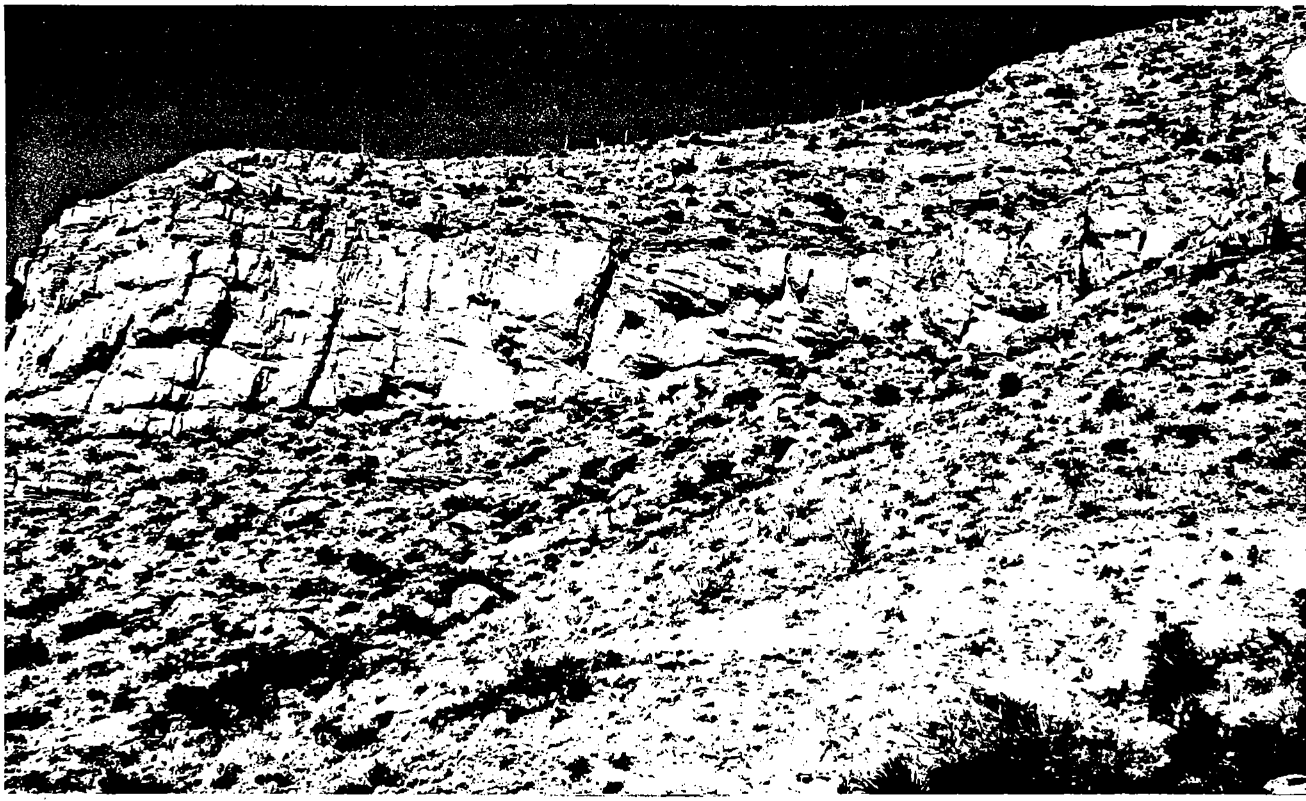


Figure 4. Precambrian rocks along I-25 east of Santa Fe.



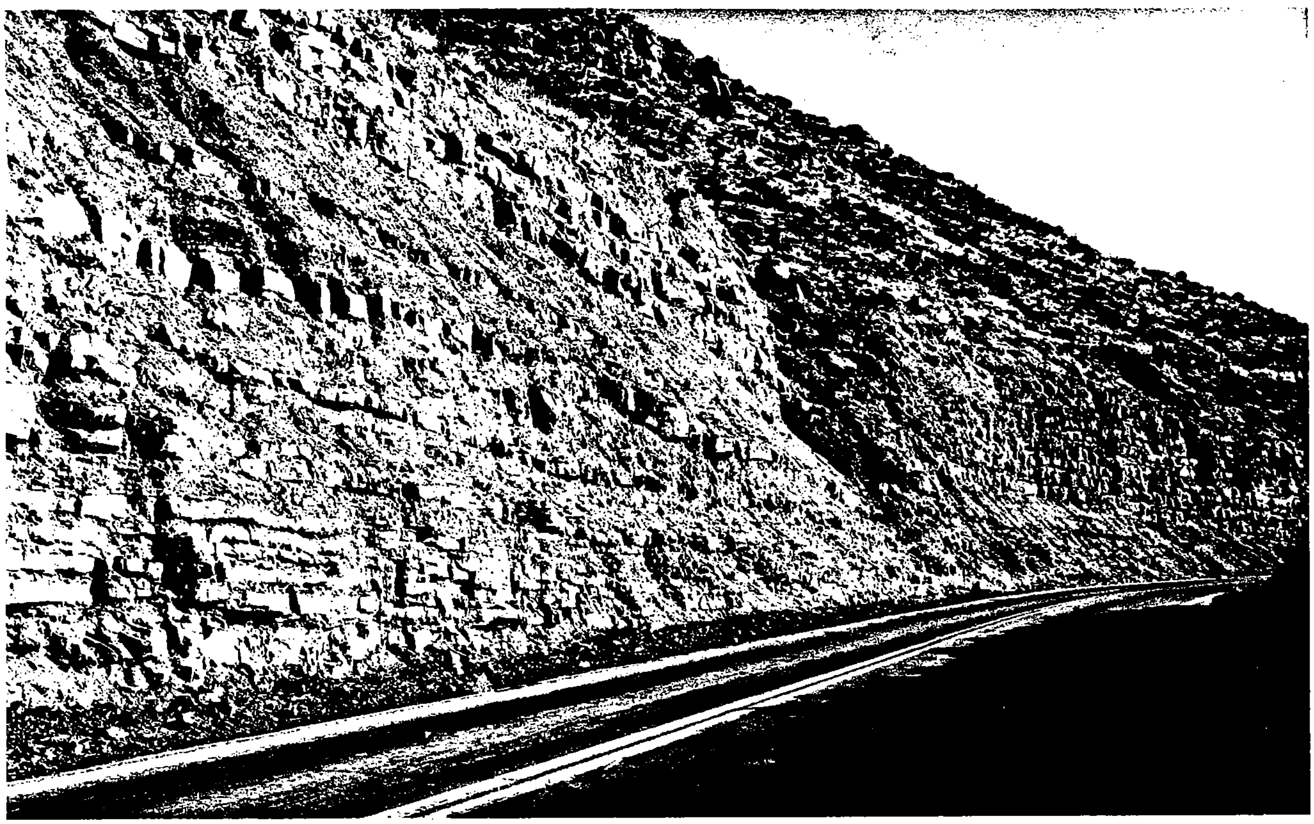


Figure 5. Thin-bedded Pennsylvanian limestone and shale (bottom); and algal mound (top) along US-82 east of Alamogordo.

Figure 6. Red sandstone and shale of Abo Formation along US-60 west of Mountainair.

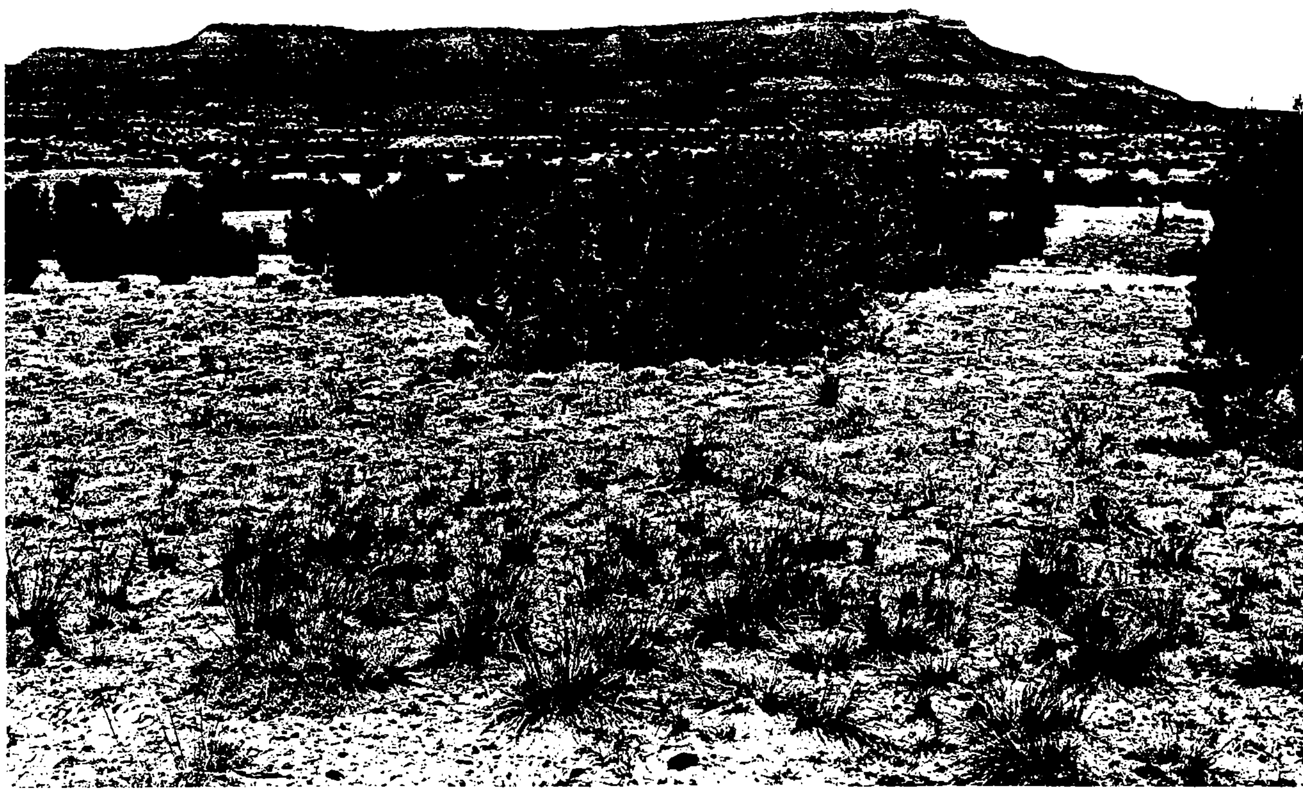


Figure 7. Glorieta Mesa, capped by Glorieta Sandstone, overlying Yeso Formation.

Figure 8. Vermejo Formation in north side of Cimarron Canyon along US-64.

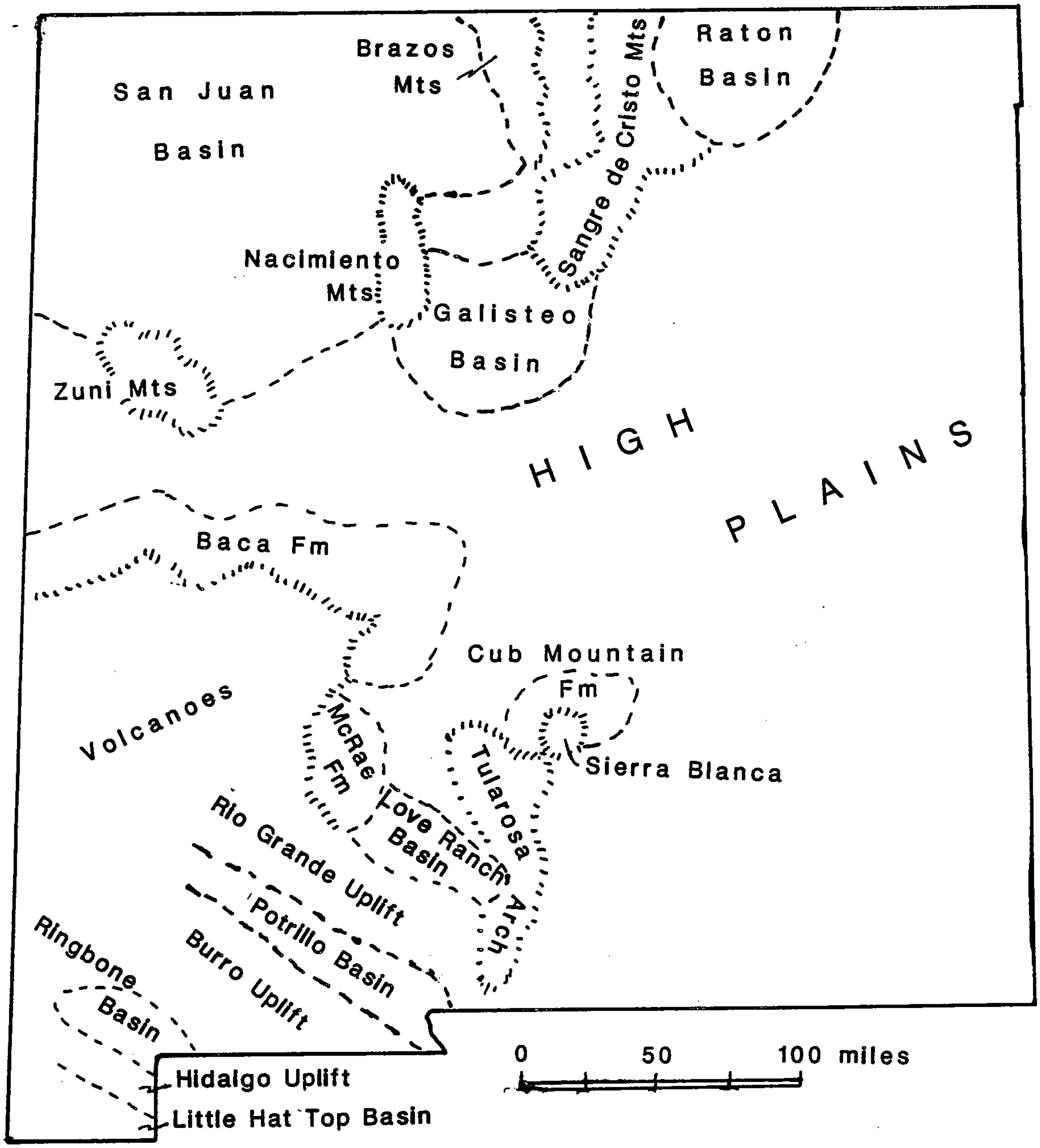


Figure 9. Paleogeographic map of New Mexico during early-middle Tertiary times.

Figure 10. Dike cutting shale beside US-82 east of Alamogordo.

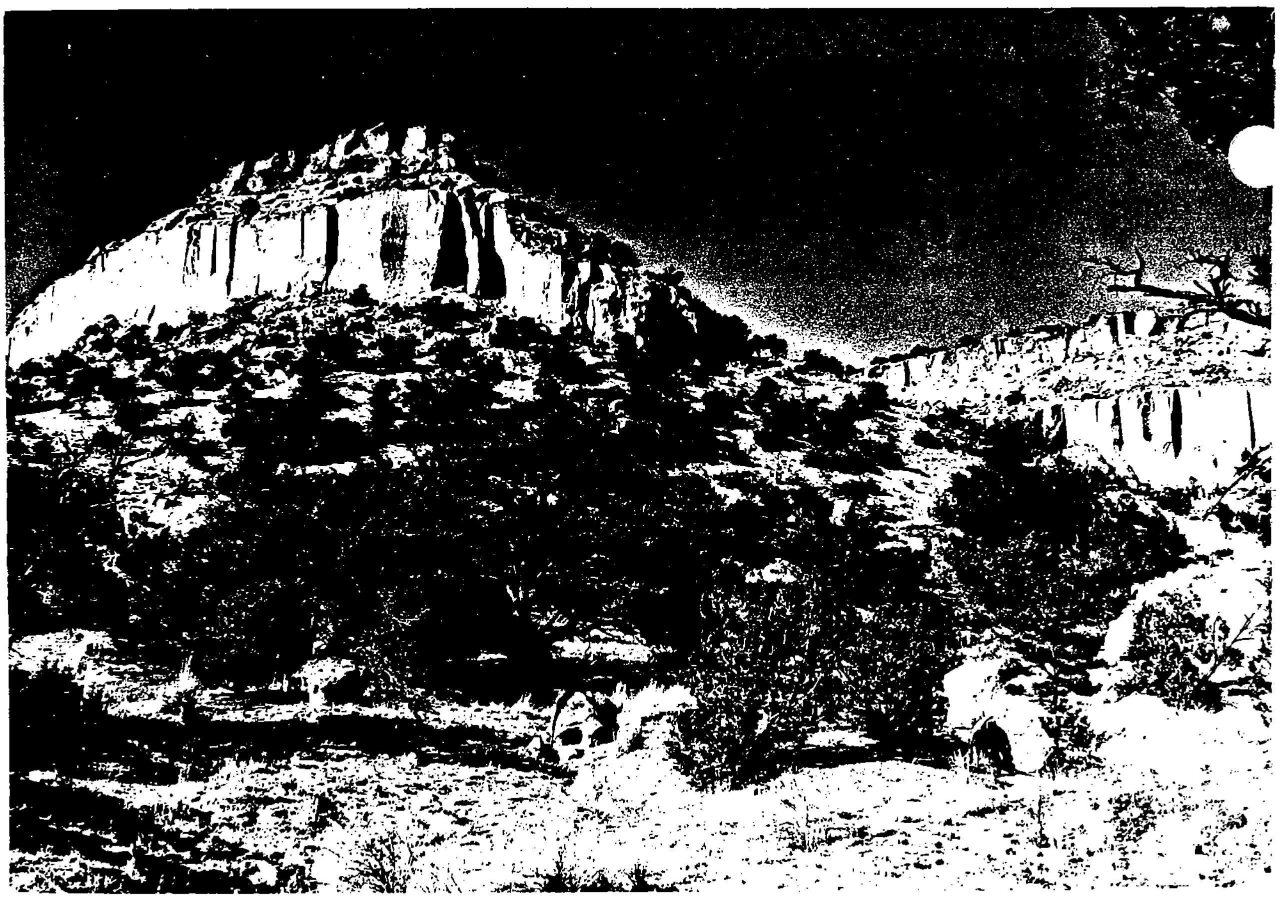


Figure 11. Bandelier Tuff southwest of Los Alamos.

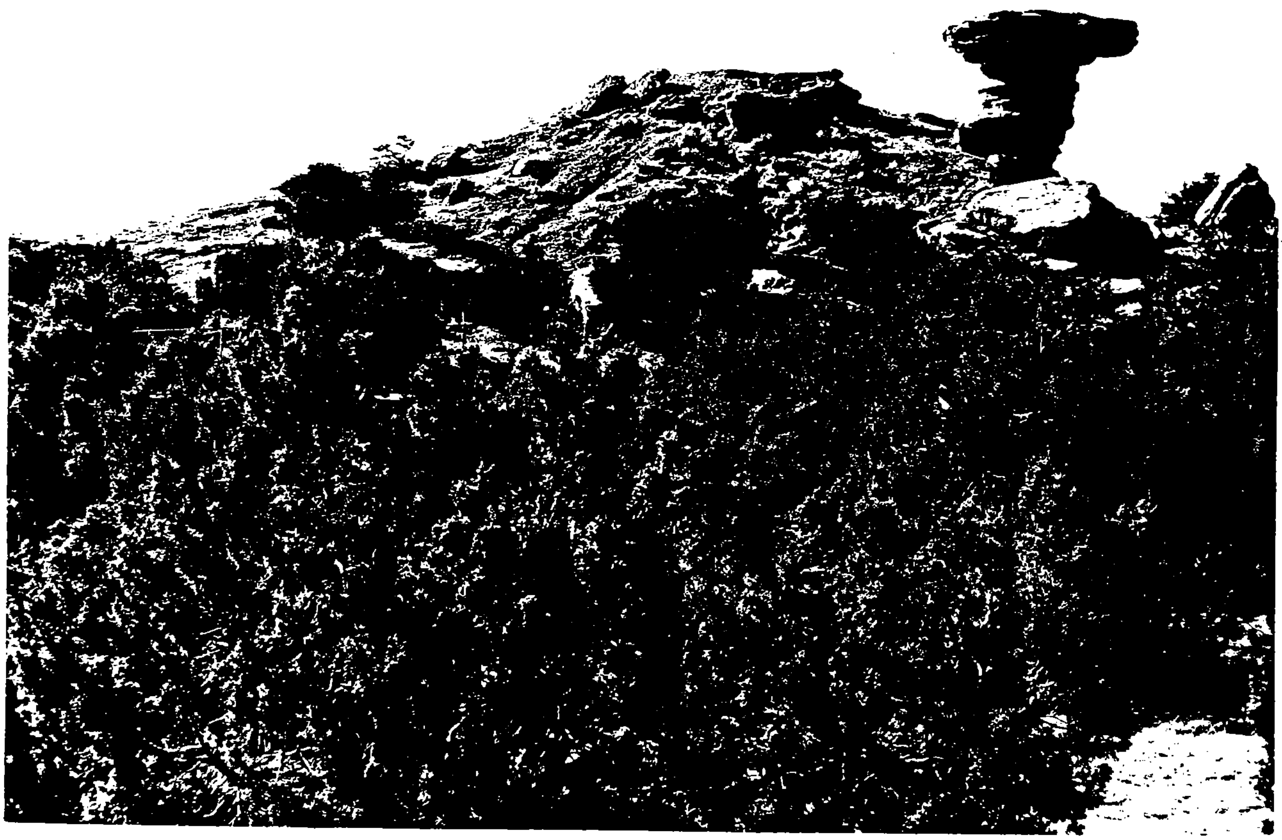


Figure 12. Camel Rock, composed of Santa Fe Group sandstones, north of Santa Fe.

Geologic Hazards of New Mexico

WILLIAM C. HANEBERG

New Mexico Bureau of Mines and Mineral Resources
New Mexico Institute of Mining and Technology
Socorro, NM 87801

Abstract– Potential geologic hazards in New Mexico include earthquakes; land subsidence due to collapsing soils, groundwater withdrawal, limestone and evaporite karst, and collapse of abandoned mine workings; earth fissures; expansive soils; slope instability; radon availability, and volcanism. As the state agency responsible by law for original investigations of geology and mineral resources in the state, the New Mexico Bureau of Mines and Mineral Resources has an active interest in the identification, assessment, and remediation of geologic hazards. Although the nature and general location of many potential hazards are well known, many detailed questions about potentially hazardous geologic processes remain unanswered. Unfortunately, a general lack of interest in the use of geologic information in planning decisions means that the state's response to geologic hazards is one of reaction rather than prevention.

INTRODUCTION

New Mexico presents engineering geologists and geotechnical engineers with a variety of geologic hazards including earthquakes, collapsible soils, subsidence due to groundwater overdraft, earth fissures, slope instability, karst, and, potentially, volcanism. This paper is intended as a generalized overview of potentially hazardous geologic processes occurring throughout the state, emphasizing work by the New Mexico Bureau of Mines and Mineral Resources (NMBM&MR) and cooperating agencies, for attendees of the 41st Highway Geology Symposium. It is not intended as a comprehensive inventory, and should not be used as a basis for planning, design, or construction.

The general geology of New Mexico, described in the previous paper by Professor Clemmons, will not be reiterated. Throughout this paper, however, reference is made to place names that may be unfamiliar to many Highway Geology Symposium attendees. Therefore, an index map of New Mexico counties and towns is provided in Figure 1.

Sources of Geologic Information in New Mexico

The primary source of geologic information about New Mexico is NMBM&MR, which is the office of the state geologist and the state agency responsible by law for original investigations of geology and mineral resources in New Mexico. NMBM&MR maintains a small environmental and engineering geology staff, which is responsible for 1) determining potential environmental impacts caused by intensive development of mineral, water, and land resources; 2) evaluating the hazard potential of natural geologic processes such as landslides, debris flows, earthquakes, and ground subsidence; 3) siting of waste management facilities; and 4) developing a comprehensive database, including reports and maps, on these subjects. Lists of publications and open-file reports, as well as the quarterly *New Mexico Geology*, are available from the NMBM&MR publications office.

Other sources of engineering geologic and geotechnical engineering information include the New Mexico State Highway Department Geotechnical Section, the New Mexico Water Resources Research Institute, the U.S. Geological Survey, the Soil Conservation Service, the State Engineer Office, geology departments and geological engineering programs as the University of New Mexico, New Mexico State University, and the New Mexico Institute of Mining and Technology (New Mexico Tech). Additional sources are included in Johnpeer (1986). The Geology Section of the New Mexico State Highway

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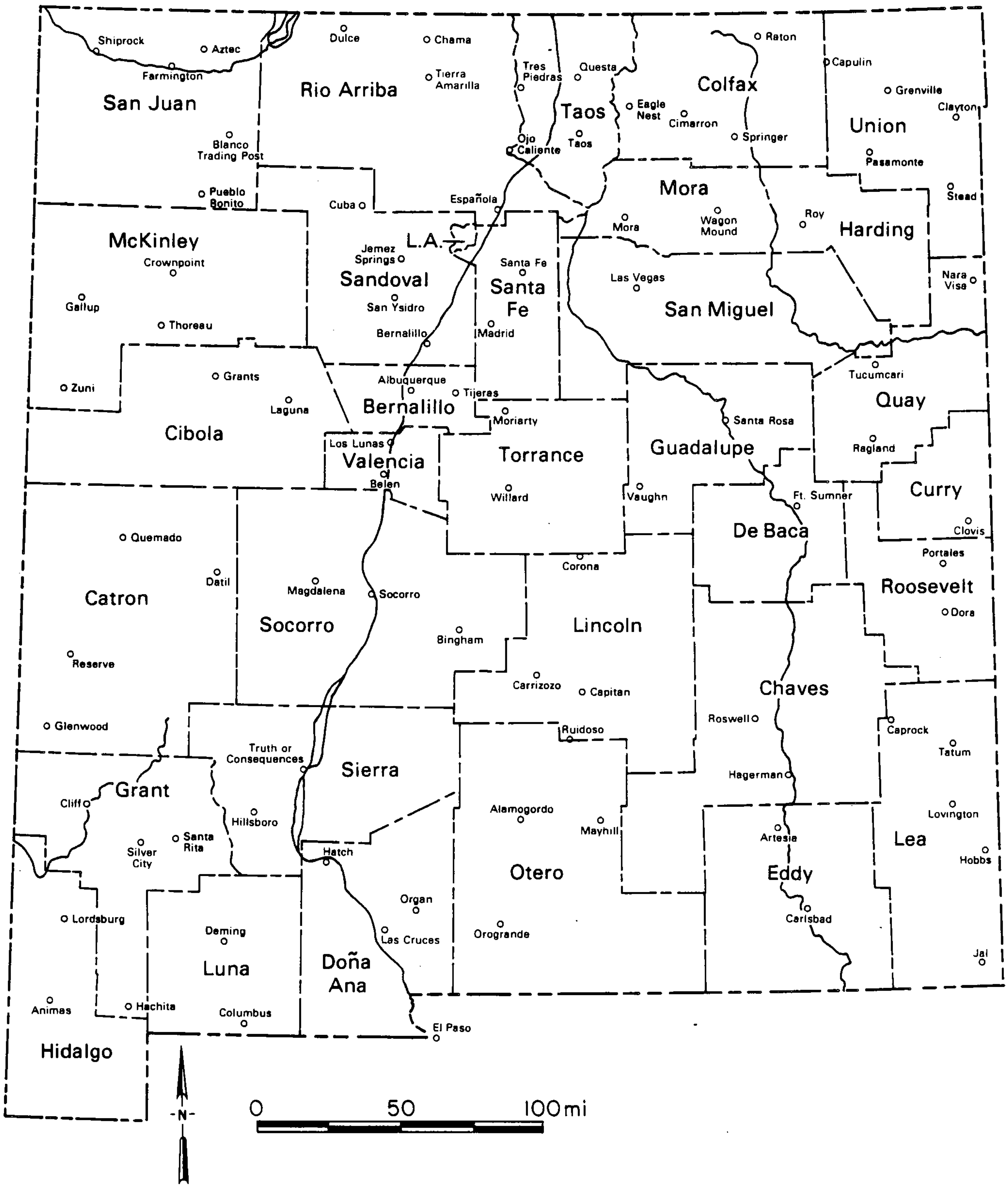

Figure 1– Index map showing counties and major towns in New Mexico.

Department has produced a series of geologic and aggregate source atlases of considerable interest to those involved in construction. Guidebooks compiled for the annual fall field conferences and other special publications of the New Mexico Geological Society constitute a valuable source of basic geologic information, as well. Sources for maps, aerial photographs, satellite imagery, thermal imagery, and radar imagery covering the state are listed in Rex and Budge (1986).

Hawley and Love (1981) present a generalized summary of geology as related to environmental concerns in New Mexico. The engineering and environmental geology of the Albuquerque area is summarized by Lambert *et al.* (1982) and Clary *et al.* (1984), and Johnpeer and Hamil (1983) provide a similar synopsis of the Socorro area. Volumes edited by Wells *et al.* (1981) and complied by Stone (1984) summarize water quality research– which is not covered in this paper– in New Mexico.

Geologic Hazards and Geologic Events

The distinction between a geologic *hazard* and a geologic *event* (*e.g.*, Rahn, 1986) is an important one. For the purposes of this paper, a geologic event is defined as the occurrence of any geologic process– perhaps a landslide. A geologic hazard, on the other hand, is defined as a geologic event which has a significant impact upon humans or their works. In many respects this distinction is analogous to the adage of a tree falling in a forest with no one to hear the sound. Thus, a large landslide in a remote, uninhabited area would be classified as an event whereas the same landslide in a heavily developed mountain resort would be classified as a hazard. It is also useful to define *potential hazards* and *potential events* as those which geologic evidence suggests may have occurred in the past, and which may occur in the future, but for which there is little or no evidence of recent activity. Examples of potential hazards or potential events might include dormant volcanoes or landslide complexes. This distinction has practical significance for states such as New Mexico, in which limited resources must be used to address specific problems rather than to compile state-wide inventories covering vast, sparsely-populated areas.

EARTHQUAKES

Historic Activity

Instrumental recording of earthquakes in New Mexico commenced with the installation of seismograph network, operated by the New Mexico Institute of Mining and Technology, as well as several other seismographs in and around the state, during the early 1960s. Seismograph networks established in northern New Mexico by Los Alamos National Laboratory in 1973 and throughout the Albuquerque-Belen Basin by the U.S. Geological Survey in 1976. Sanford *et al.* (1981) reported a total of 247 earthquakes with magnitude $M > 1.5$ recorded in New Mexico between 1962 and 1980, with many hundreds more of lesser magnitude. These earthquakes were clustered along the Rio Grande Rift, the High Plains, the Jemez Lineament, and the margin of the Colorado Plateau. Sanford *et al.* (1981) speculate that earthquakes in the High Plains of southeastern New Mexico, which were unknown before 1964 but occur in an area of several major oil fields, may have been induced by hydrocarbon recovery practices. Similar occurrences have been reported throughout the world (Segall, 1989), and can be shown to be both empirically and theoretically related to *reduction* of pore pressures (Segall, 1985, 1989). Recent seismic activity has included a swarm of more than 20 earthquakes of $2.0 < M < 4.7$, with epicenters between Belen and Socorro, between late November 1989 and early February 1990 (A.R. Sanford, personal communication, 1990). These shocks, like other historic activity throughout the area, are characterized by swarms apparently related to the injection of a mid-crustal magma body and attendant crustal deformation near Socorro (Sanford *et al.*, 1981; also see Larsen and Reilinger, 1983 and Sanford, 1983).

Based upon extrapolation of instrumental data collected between 1962 and 1975, Sanford *et al.* (1981; also see Sanford *et al.*, 1972) predict one earthquake of $M = 4.8$ every 50 years and one earthquake of $M = 5.1$ every 100 years. Similar estimates for the Los Alamos area suggest an earthquake of $4.5 < M < 5.0$ with an epicenter within 70 km of Los Alamos once every 100 years (House and Cash, 1988). Algermissen and Perkins (1976) used historical seismicity and identification of earthquake source areas to estimate maximum horizontal acceleration of 0.08 g, where $g \approx 9.81$ m/s is gravitational acceleration, in rock through most of the Rio Grande Rift, with 90% probability of not being exceeded in 50 years. However, maximum horizontal acceleration estimates peak at 0.30 *g* between Belen and Socorro, and 0.15 *g* near Silver City. Outside of these areas, maximum probable horizontal acceleration is less that 0.04 *g*.

Reservoir-induced seismicity has been documented at El Vado and Heron reservoirs, near Chama, in north-central New Mexico (El-Hussain and Carpenter, 1990) between 1976 and 1984. Maximum magnitude was $M = 2.7$, with a sharp drop in the frequency of higher-magnitude earthquakes. Although El-Hussain and Carpenter correlated reservoir filling with episodes of increased earthquake frequency, they did not investigate focal mechanisms of these earthquakes, which must be considered before analyzing time lags between filling and seismicity (Roeloffs, 1988). As such, the details of reservoir-induced seismicity in the El Vado and Heron area remain unknown.

Although earthquakes have been recorded instrumentally for only the past few decades, written description of seismic activity date back as far as the Territorial period of the middle 19th century. Northrop (1976) found descriptions of 1111 earthquakes in New Mexico during the period 1849-1975, of which 523 descriptions or recordings were definite and 588 were less definite or vague. Of those earthquakes described with certainty, 508 had epicenters in New Mexico and 3 were produced by atomic testing, including the explosion of the first atomic bomb at the Trinity Site. Northrop (1976) states that 76% of the earthquakes with New Mexico epicenters occurred along the Rio Grande Rift. As noted by Sanford *et al.* (1981), though, this figure is most likely an artifact of settlement patterns. Well-documented, pre-instrumentation swarms along the rift occurred in 1849-1850, 1893, 1904, 1906, and 1935, with virtually all earthquakes of Modified Mercalli Intensity of VII or less (Northrop, 1976).

Paleoseismicity and Long-Term Earthquake Hazards

Geomorphic studies of young fault scarps suggest that estimates of seismicity based upon instrumental observation may significantly underestimate long term earthquake hazards in New Mexico. Without the advantage of modern scarp-dating methods, Sanford *et al.* (1972) estimated a recurrence interval of 1000 years for earthquakes as large as $M = 7.6$ and 100 years for earthquakes as large as $M = 6.6$. More recently, Machette (1986, 1988) used soil geomorphology and slope degradation models to estimate a recurrence interval of 6000 to 7500 years for $M = 7.0$ Pleistocene and Holocene earthquakes along the La Jencia fault near Magdalena, in Socorro County. In a similar study, Machette (1987) estimates an area-wide recurrence interval of some 2000 years for $7.0 < M < 7.5$ earthquakes in the vicinity of the White Sands Missile Range. Foley *et al.* (1988) estimate a maximum credible earthquake of $M = 7.25$ along the Caballo fault near Elephant Butte and Caballo Reservoirs in Sierra County. Menges (1987, 1988) likewise estimates magnitudes on the order of $M = 7.0$ for Quaternary and Holocene earthquakes along the western Sangre de Cristo front, with uncertain recurrence intervals. Gile (1987; also see Machette, 1987) describes Late Holocene movement, perhaps as recently as 1000 years ago, along the Organ Mountains fault near Las Cruces. Although he does not estimate magnitudes, Gile states that movement "must have been accompanied by severe and extensive earthquake activity," and Beehner (1990) cites an unpublished maximum credible earthquake of $M = 7.5$.

There is in general little public concern about earthquake hazards in New Mexico, probably due to low levels of historic earthquake activity, and, therefore, little effort has been invested in earthquake hazard studies. This lack of foresight is particularly acute along the Rio Grande valley, including parts of Albuquerque, where liquefaction of saturated alluvium is likely to present a hazard during strong earthquakes. The only state-wide earthquake hazard assessment if that of Johnpeer *et al.* (1986), who used distributions of historic earthquake damage, young volcanic rocks, young faults, probabilistic horizontal acceleration, shallow ground water, and major lifelines to produce a generalized earthquake hazard map. According to their work, earthquake hazards are very low to low throughout most of New Mexico; moderate along the Rio Grande Rift, southwestern, and south-central mountains; and high only in populated areas, as expected from the definition of a geologic hazard. Gardner and House (1987) produced a similar series of hazard maps showing geology, geomorphic surfaces, surface rupture potential, zones of potentially hazardous mass-wasting, and culture in the Los Alamos area.

LAND SUBSIDENCE

Collapsible Soils

Collapsible soils, which lose a significant amount of volume through reduction of porosity when wetted, have damaged roads and structures in several parts of New Mexico. Research by NMBM&MR (Reimers, 1986; Johnpeer *et al.*, 1985a,b; Shaw and Johnpeer, 1985a,b), prompted largely by damage to single-family dwellings near Espanola, New Mexico, and by the New Mexico State Highway Department (Lovelace *et al.*, 1982) has produced a large body of information concerning the identification and avoidance of collapsible soils. In addition to the Espanola-Santa Fe region, collapsible soils are known to have caused damage in Albuquerque, Alamogordo, Belen, and Socorro.

Collapsible soils are typically composed of poorly-sorted sand, with minor amounts of clay or evaporites, deposited by debris flows (incorrectly referred to as mud flows by some; see Johnson, 1984, for a review of terminology) on gently-sloping alluvial fan complexes. Before collapse, soil structure is porous and open, with sand grains supported by point-to-point contacts and bound by thin veneers of clays or, in some cases, evaporites. In many cases collapsible soils can be identified in the field by their geologic setting, low unit weights (approximately 12,000 to 16,000 N/m^3) or high void ratios (approximately 0.7 to 1.0), and disaggregation when immersed in water. Standard penetration test blow counts, however, are not an effective method of collapsible soil identification, as these soils are typically strong when dry. Laboratory tests for collapsible soils include a modified consolidation test, in which a dry sample is subjected to its original confining pressure, plus any structural loads, and then saturated for 24 hours. The collapse potential of the soil is then given by:

$$CP = \Delta e/(1 + e_0)$$

where Δe is the change in void ratio after saturation and e_0 is the original void ratio. Values of $CP < 1\%$ generally are taken to signify little or no loss of volume upon wetting, whereas values in the range of $1\% < CP < 20\%$ or more are taken to signify slight to significant loss of volume upon wetting (*e.g.*, Reimers, 1986, p. 71).

Collapsible soil deposits can extend to tens of meters in depth and remedial measures can be prohibitively expensive, so these problem soils are best identified before design and certainly before construction. Pre-construction treatments include removal of shallow deposits, injection of water to induce collapse (*e.g.*, Shaw and Johnpeer, 1985b), dynamic compaction, vibrofloatation, deep plowing, and flooding (Lovelace *et al.*, 1982). Heavily reinforced concrete slabs, cast-in-place concrete piers or belled caissons, or driven piles can also be used to support structures on collapsible soils (Curtis and Toland, 1964; Shaw and Johnpeer, 1985b). Elimination of water sources, including sewer or water line leaks, lawn watering, and septic tanks can reduce the potential for collapse if problems are discovered after construction. Although collapsible soils can be detected during preliminary geotechnical investigations, well before design and construction, they promise to remain a hazard as mountain-front communities continue to grow unchecked.

Collapse of Abandoned Mine Workings

Some damage to roads and structures due to the collapse of abandoned coal mines in the Cretaceous strata of the San Juan basin, in northwestern New Mexico, has been reported (D.W. Love, personal communication, 1990). However, these problems have never been adequately described in the geologic literature.

Groundwater Overdraft

Prudent regulation of groundwater consumption, particularly for irrigation, has allowed New Mexico to avoid the severe land subsidence and fissuring problems encountered in Arizona, California, and Nevada. The only area of documented land subsidence associated with groundwater overdraft in New Mexico is in the Mimbres Basin near Deming, in Luna County (Contaldo, 1989). Heavy agricultural demand since the turn of the century has produced a basin-wide cone of depression, with maximum water level declines of some 30 m recorded south of Deming. Level lines across the Mimbres Basin have never be re-surveyed, so it is impossible to accurately estimate the magnitude of subsidence. However, comparison of changes in the elevation of isolated benchmarks and protruding well casings suggest maximum subsidence on the order of 0.30 to 0.35 m near the center of the basin (Contaldo, 1989). Evaluation of subsidence potential is not routinely incorporated into groundwater resource studies in New Mexico, for example in the rapidly expanding Albuquerque and Las Cruces, New Mexico - El Paso, Texas areas. This is probably due to the lack of documented subsidence problems. One notable exception is a study by MacMillan *et al.* (1976), who evaluated the potential for land subsidence associated with the withdrawal of large amounts of saline groundwater from the Tularosa Basin in south-central New Mexico.

EARTH FISSURES

Associated with Groundwater Overdraft

A particularly striking form of ground failure is the development of large, open earth fissures, which have been reported in many areas of the Southwest (*e.g.*, Holzer, 1984). Contaldo (1989; also see Contaldo and Mueller, 1988) describes 12 occurrences of earth fissures in silty to gravelly basin-fill deposits in the Deming area, all within the cone of depression produced by groundwater overdraft. In some cases the fissures form polygonal or orthogonal networks, whereas in other cases the fissures appear as curvilinear features several hundreds of meters long. Although the fissures first appear as hairline cracks, they can be widened to several meters in width due to subsequent erosion and piping during heavy storms. The Deming fissures occur in sparsely-populated range land so, although one fissure passes within several meters of an occupied mobile home, no structural damage has been reported. Nonetheless, the presence of large, open cracks presents both a hazard to children, cattle, and vehicles.

Other Occurrences

Although earth fissures in the Deming area, as well as many other areas described by Holzer (1984), are clearly associated with groundwater overdraft, other fissures are located in areas where groundwater withdrawal can be safely eliminated as a possible cause. Sanford *et al.* (1982, 1983) studied a 1400 m long earth fissure which appeared in valley-fill sands and gravels along the Rio Grande Rift near San Marcial, New Mexico during construction of Interstate 25 in 1981. The fissure could not be attributed to any known seismic activity and, on the basis of regional geophysical and tectonic data, Sanford *et al.* (1982, 1983) suggested that the fissure may have been the result differential compaction of unconsolidated deposits over a buried fault scarp. Similar fissures, with no apparent relationship to groundwater withdrawal, have been reported elsewhere. For example, Bell and Hoffard (1990) and Bell *et al.* (1989) describe fissures in Nevada, which probably formed in response to aseismic creep along known faults. Keaton and Shlemon (1990) describe a network of earth fissures at a proposed low-level radioactive waste disposal site West Texas; although they are unable to determine the origin of the fissures with any certainty, Keaton and Shlemon suggest that regional uplift is one possibility.

Current Research

Current research at NMBM&MR is concentrated on geophysical characterization and mathematical analysis of soil deformation fields associated with earth fissures. Work completed to data shows that earth fissures near Deming (Haneberg, 1990a) and San Marcial (Haneberg *et al.*, in review) are associated with readily-identifiable buriea structures and distinctive P-wave velocity anomalies. Results of the seismic surveys are also being used to constrain mechanical models of elastic stress and displacement fields, modified from models of monoclinal flexuring in the structural geology literature (Sanford, 1959; Reches and Johnson, 1978), in soil layers deforming under their own weight (Haneberg *et al.*, in review). It is hoped that future work will include monitoring of surface and subsurface displacement, seasonal water level fluctuations, and seasonal temperature fluctuations as the basis for further mechanical analyses, including the effects of thermoelastic stresses and vadose zone seepage forces.

EXPANSIVE SOILS

Expansive soils of the order Vertisol present foundation design problems in urbanized areas such as Albuquerque and Las Cruces (Hacker, 1977; Bulloch and Neher, 1980). Gustavson (1975) reviews identification microtopographic features (gilgai) and engineering problems associated with vertisols along the Texas coastal plain, and much of his work can be applied to New Mexico, where the most severe problems are caused by smectitic soils such as those of the Armijo Series. Armijo soils form in alluvial loams and clays deposited along the Rio Grande floodplain and the clays, in particular, have high shrink-swell indices.

In other cases, smectitic shale bedrock has caused highway construction problems, for example in Cretaceous Morrison Formation shales excavated along Interstate 40 between Santa Rosa and Tucumcari (Kelley and Kelley, 1972). This section of highway was graded, rolled, and sealed with an asphalt membrane to prevent infiltration.

Geotechnical exploration procedures in areas where expansive soils may be a problem should include a review of county soil surveys and analysis of clay mineralogy in order to estimate the general potential for shrink-swell problems. Once areas of expansive soils have been identified, laboratory testing of shrink-swell potential can yield quantitative information useful for foundation design. Remedial measures are similar to those for collapsing soils, including removal of shallow problem soils, pre-compaction or pre-wetting, deep foundations, and elimination of moisture sources (Das, 1984, p. 460-474).

SLOPE INSTABILITY

Evidence for Past Slope Stability Problems

Slope instability is not currently a significant geologic hazard in most parts of New Mexico, although landsliding is a persistent problem along some highways in the northern mountains. Regional 1:500,000 landslide inventories based upon aerial photograph and topographic map interpretation (Cardinali *et al.*, 1990; Guzetti and Brabb, 1987), however, show evidence of past slope instability throughout much of New Mexico. In many cases, the evidence is limited to nearly vertical escarpments subject to rockfalls and topples. In other cases, evidence of large landslide and debris flow deposits were identified.

Evidence of past slope instability is one of the key elements in the delineation of potentially hazardous areas (Nilsen, 1986; Hansen, 1984); therefore, even if slope instability is not currently a significant hazard, the possibility should be incorporated into long-term planning decisions in areas of prior instability. Reactivation of large landslide complexes in Utah during the early to middle 1980s, for example, has been attributed to a relatively sudden and unprecedented reversal of a long-term drying trend (Fleming and Schuster, 1985). Similarly, Watson and Wright (1963) use pollen analysis to infer a colder, wetter Pleistocene climate during movement of large slump blocks capped by resistant Miocene sandstone (locally referred to as toreva blocks) in northwestern New Mexico. Costs associated with the reactivation of large landslide complexes are sobering: Fleming *et al.* (1988), for example, estimate total costs of nearly $2 million to repair damage and prepare for future movement when the Manti landslide in Utah, some 6.5 km from the nearest dwelling, was reactivated in 1974.

Slope Instability and Human Activity

Human activity, especially road building, can also result in reactivation of landslides. Bennett (1974) describes widespread slope stability problems in nearly saturated landslide debris, derived from Pleistocene outwash deposited over Cretaceous Mancos Shale, along 30 km of U.S. 64 between Tierra Amarilla and Tres Piedras, in eastern Rio Arriba County. These failures are concentrated in cut and fill slopes along the right of way, and Bennett notes that only preliminary geologic studies, with no provisions for slope stability analysis, were conducted prior to construction. Another reactivation of an obvious landslide complex occurred in Cretaceous shales and limestones capped by Quaternary basalts during construction of Interstate 25 between Springer and Raton, in Colfax County (Lovelace, 1976). A large landslide, apparently reactivated due to seepage from abandoned mine workings, affected a number of homes near the Taos ski valley in 1979; remedial measures included installation of horizontal drains and diversion of surface water, which increased the pre-slide factor of safety of 0.92 to 1.28 (Bennett, 1979).

Potentially hazardous slope failures can also occur in areas with no evidence of past instability. NMBM&MR is currently investigating a recent irrigation-induced debris flow that was mobilized from a sequence of Quaternary stream terrace sands, gravels, and cobbles east of Espanola, near the town of Cordova, in Rio Arriba County (Haneberg, 1990b). Neither the Guzetti and Brabb (1987) regional map nor visits to the site suggest evidence of past slope instability in the area. Failure was apparently due to long-term accumulation of perched water along one of several fine-grained, buried soil horizons within the coarse-grained terrace deposits. Gradient reversals and small leaks in an unlined irrigation ditch above the failure suggest that progressive failure had been occurring for some time, although the pre-dawn debris flow *per se* was rapid. The volume of debris is not great, on the order of 10^4 m^3, and the only damage was burial of an unimproved dirt road, diversion of a small stream, and loss of pasture land. However, the existence of untold miles of community irrigation ditches, many dating back several centuries, in steep terrain makes future irrigation-induced failures in northern New Mexico a potential hazard.

LIMESTONE AND EVAPORITE KARST

Sweeting (1972) describes solution-subsidence relief near Santa Rosa, which she attributes to dissolution of both limestone and gypsum in Permian San Andres Limestone and the collapse of overlying Santa Rosa Sandstone. Kelley (1972) also describes as limestone karst as a minor geologic hazard in the area. Hill (1987) describes the geologic evolution of Carlsbad Cavern, which is more a tourist attraction than a geologic hazard, and uses geochemical evidence to suggest that the caverns were dissolved by sulfuric rather than carbonic acid. The source of the necessary sulfur seems to have been H_2S leaking from nearby petroleum reservoirs.

A problem much more prevalent than limestone karst is the dissolution of Permian evaporites– predominantly halite– and collapse of overlying strata, which has been occurring along the Pecos River valley since Pliocene time (Osterkamp *et al.*, 1987, p. 193; Gustavson and Finley, 1985; Bachman, 1974, 1984). Simpkins *et al.* (1981) describe the development of sinkholes, closed depressions, extensional fractures, and faults associated with evaporite dissolution in the Texas Panhandle and neighboring parts of New Mexico, as well as damage to highways, stock tanks, and reservoirs. Remedial measures include filling of depressions in asphalt pavements with additional asphalt, and mud-jacking rigid concrete pavements such as Interstate 40.

Evaporite dissolution has also been an issue in the siting of the Waste Isolation Pilot Plant (WIPP) near Carlsbad, where transuranic nuclear waste will be stored in Permian salt beds. Anderson (1981), for example, believes that the presence of a structurally-complex disturbed zone beneath part of the WIPP is evidence for early stages of salt dissolution at the site. Davies (1989) reviews a number of locations with evidence for large-scale evaporite dissolution; including a sinkhole in western Texas, two collapse chimneys in southeastern New Mexico, and a structural depression beneath the WIPP site; and proposes a generalized hydrologic model for dissolution of a salt layer underlain by a leaky aquifer. He also suggests that deformation related to evaporite dissolution can be characterized by two end-member, ductile downwarping and brittle collapse, that are functions of deformation rates. Other salt-related problems investigated during the course of WIPP development include estimation of seepage rates into storage rooms excavated in salt (McTigue, 1989a) and the effects of excavation-induced permeability increases near storage rooms on seepage rates (McTigue, 1989b).

RADON AVAILABILITY

The widespread occurrence of uraniferous sedimentary and igneous rocks makes inhalation of radon gas a potential hazard in many parts of the state. As part of a preliminary assessment, McLemore and Hawley (1988) used aerial radiometric surveys to compile radon availability ratings for the 33 counties in New Mexico. Other geologically-oriented radon studies are described in a theme section of the May, 1990 issue of *Geophysical Research Letters*. For example, aerial radiometric surveys have also been used to delineate areas of high radon concentration in California (Wollenberg and Revzan, 1990) and the Pacific Northwest (Duval and Otton, 1990); in particular, Duval and Otton found a reasonably good correlation between surface radium and indoor radon in cases where radon concentration was less than 2 pCi/liter. The physics of radon transport through soils is analyzed by Owczarski *et al.* (1990), who model the effects of water content and atmospheric pressure on radon migration through different soil types, and Narasimhan *et al.* (1990), who model the effects of sinusoidally-varying atmospheric pressure on radon concentration near a hypothetical basement.

Radon availability ratings in the McLemore and Hawley (1988) study of New Mexico were defined as follows:

High radon availability– Greater than 2.7 ppm uranium on aerial radiometric maps and, in general, well-drained and permeable soils. Counties falling into this category are Dona Ana, Hidalgo, Los Alamos, Luna, McKinley, Rio Arriba, Sandoval, Santa Fe, Socorro, and Taos.

Moderate radon availability– 2.3 to 2.7 ppm uranium on aerial radiometric maps and, in general, moderately permeable soils. Counties falling into this category are Bernallilo, Catron, Cibola, Chavez, Colfax, Eddy, Grant, Lea, Lincoln, San Juan, Sierra, Quay, and Union.

Low radon availability– less than 2.3 ppm uranium on aerial radiometric maps and, in general, low permeability soils. Counties falling into this category are Curry, De Baca, Guadalupe, Harding, Mora, Otero, Roosevelt, San Miguel, Torrance, and Valencia.

It is important to emphasize that these ratings are preliminary estimates of radon *availability* averaged over entire counties, and not the *actual* radon concentrated within a given building at a given time. Therefore, it is possible for homes in high-availability counties to have low indoor radon values, and for homes in low-availability counties to have high indoor radon values. Site-specific details such as soil type and moisture content, soil or bedrock fracture systems, foundation type, ventilation, and time of year can all produce widely-scattered indoor radon readings in nearby buildings.

VOLCANISM

The study of Cenozoic volcanism in New Mexico has produced an enormous body of literature, much of which is summarized in Chapin and Zidek (1989). Quaternary volcanic rocks, the product of at least 700 eruptions over the past 5 Ma (Limburg, 1990), cover large areas of New Mexico. Although most of these have been effusive basaltic eruptions, presenting little danger beyond the loss of property, the presence of the rhyolitic Jemez volcanic center in north-central New Mexico suggests that the possibility of violent plinian eruptions cannot be precluded. Moreover, large snowpacks in mountainous areas such as the Jemez could give rise to catastrophic debris flows in the event of a winter or spring eruption.

Because much of the Quaternary basaltic volcanism has occurred along the Rio Grande rift, which also contains much of New Mexico's population and infrastructure, small basaltic eruptions might cause closure of roads, damming of rivers and irrigation ditches, and loss of structures. However, even if it is assumed that any eruption would affect the entire state, the eruption data tabulated by Limburg (1989) suggest that the state-wide recurrence interval is on the order of of 10^4 years. The historical predominance of basaltic eruptions further implies that the recurrence interval for explosive rhyolitic eruptions is higher. Therefore, volcanism in is a long-term hazard of consequence to only the most sensitive facilities– for example national laboratories or nuclear waste disposal sites.

SUMMARY

Potential geologic hazards in New Mexico cover a broad spectrum, from earthquakes to collapsing soils to landslides. No estimates of economic impact are available, but collapsible soils are probably the most significant *actual* geologic hazard in the state. However, the most significant *potential* hazard is probably a large earthquake on the order of magnitude $M = 7$, with an estimated recurrence interval on the order of 10^3 years. Recent experiences in Utah have shown that landslide and debris flow hazards, along with damage costs, can also be expected to increase significantly during wet periods. A wealth of unanswered questions concerning the distribution, mechanisms, and mechanics of hazards makes New Mexico fruitful ground for basic and applied research on potentially hazardous geologic processes.

Hazards brought on by the action of man– such as collapsing soils, subsidence, reactivation of landslides due to construction– can be predicted routinely during preliminary geotechnical investigations. There is in theory, therefore, no reason why these sorts of potential hazards could not be virtually eliminated in New Mexico. Other hazards, such as earthquakes and, perhaps, volcanic eruptions, can only be anticipated and prepared for, making reduction of damage somewhat more difficult. Unfortunately, many New Mexicans have yet to grasp the fact that geologic hazard assessment can be an integral part of prudent and sustained growth. As such, the role of agencies such as NMBM&MR will be largely one of reaction rather than prevention into the foreseeable future.

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HIGHWAY DAMAGE RELATED TO A FAULT NEAR PIERRE, SOUTH DAKOTA

By Donley S. Collins and Henri S. Swolfs
U.S. Geological Survey, MS 966, Denver Federal Center, Box 25046, Denver, CO 80225

ABSTRACT

A northwest-striking fault exposed in a highway cut along U.S. Highway 14, about 18 mi west of Pierre, S. Dak., has been actively displacing the roadway since construction of the cut in 1983. This fault movement is confined to the cut in the Virgin Creek Member of the Pierre Shale and manifests itself as a recurring west-facing bump in the roadway after the surface is scraped and repaved. To better understand the physical properties of the host rock and fault gouge, a 6-ft-deep trench was excavated on the south side of the highway across the fault trace. Samples were collected at regular intervals along the trench and analyzed for clay mineralogy and chemistry, bulk density, and moisture content. The fault gouge contains fewer expandable clays and has higher density and moisture content than the adjacent host rock. The fault gouge also is a compacted, slickensided, fissile, and permeable to infiltrating surface water. During the 6 hrs the trench remained open, more than 2 in. of water collected on the floor of the trench, indicating the proximity of a water table. Geodetic data, contemporary seismicity, and studies of stream morphology in the surrounding region indicate that the fault-induced road damage probably is not related to recent tectonic activity. We believe that the removal of about 37,000 yd^3 of soil and rock during excavation of the cut contributed to the reactivation and rebound of the fault, to the dilation of fractures along the fault zone, the encroachment of meteoric water into the fault zone, and to an increased susceptibility of expandable clays to swelling; a similar rebound process was proposed by the U.S, Army Corps of Engineers during excavation for the Oahe Dam Stilling Basin in the Pierre Shale.

INTRODUCTION

While studying the physical properties of the Pierre Shale, it was discovered that faults within the project area were more extensive than first realized and that several of these faults showed recent, recurrent vertical movement of limited lateral extent that resulted in highway damage. This type of highway damage has been recognized since 1952 at several locations in South Dakota.

Geologic field work was begun in 1986 to identify, describe, date, and map the distribution of exposed faults within a 648-mi^2 study area located west of Pierre, S. Dak. (fig. 1). The purpose of this paper is to describe an occurrence of highway damage associated with fault movement without any apparent current tectonic activity, and to suggest possible means so to avoid or to correct for fault-related highway damage within the project area.

PREVIOUS WORK

Highway pavement failures associated with faults have been reported for a number of South Dakota highways, including both old and new U.S. Highway 14 (Crandell, 1958; Hammerquist and Hoskins, 1969) and State Highway 1806. As early as 1952, Crandell (1958) noted several U.S. Highway 14 failures that consisted of swells or sags of a few inches to as much as a foot that extended over distances of 10 to 100 ft along the axis of the highway. Crandell concluded that faults and (or) bedding surfaces that dip toward the road grade provide zones of permeability in the otherwise impermeable Pierre Shale. Thus, most failures probably are related to the accumulation of excessive amounts of moisture in the materials underlying the highway; these materials would become plastic and cause failure of the road surface under traffic. "Under traffic" implies that highway deformation is a result of compaction of the plastic material.

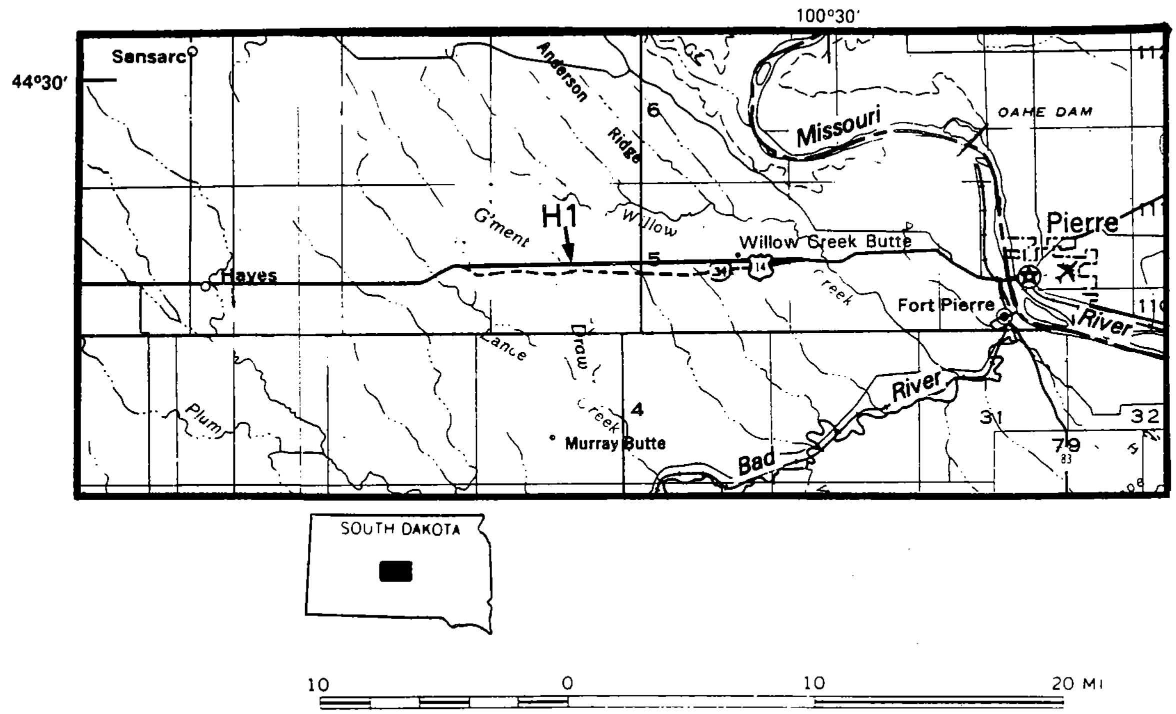

FIGURE 1. Map showing location of fault-related highway damage in a cut along U.S. Highway 14 (site H1) within the Stanley County, S.Dak., study area. The abandoned section of U.S. Highway 14 is shown by a dashed line.

Hammerquist and Hoskins (1969), in a study of roadway bumps along Interstate 80, suggested that differential uplift created by "more and faster" swelling clay in the fault gouge than in the surrounding Pierre Shale is the cause for the most severe bumps along highways constructed in the Pierre Shale. Along a fault zone, water encroachment can swell a 1/2-in.-thick gouge to a thickness of as much as 6 in. Dilation of fractures due to removal of overburden also allows cyclic wetting and drying that break the gouge into finer grains, thus making the gouge more susceptible to rapid swelling than the surrounding shale (Hammerquist and Hoskins, 1969). In comparison, the intact shale on either side of the fault zone does not break down as much during wetting and drying cycles, and therefore remains less fragmented than the gouge. To contrast the amount of water found near a fault to that present in the surrounding host rock, Hammerquist and Hoskins (1969) conducted crude infiltration tests and showed the in-situ permeability to be as much as 30 times greater in fractured shale near a fault than in relatively unfractured shale only 35 ft away. Without citing or presenting X-ray clay-mineralogy data to support the belief of the presence of a high-swelling-clay content for the shale gouge, they speculated that if the gouge has a montmorillonite and (or) illite composition, a 1/2-in. layer of dry gouge will become 5 in. of wet gouge resulting in swelling pressures of as much as 15 tons/ft^2. They concluded that swelling soil could easily lift 9 in. of concrete pavement having a few feet of base course.

Hammerquist and Hoskins (1969) showed that swelling fault gouge could produce highway bumps from 1 to 4 yrs after construction and that some swelling fault gouge could produce continuing road deformation for the lifetime of a pavement. They also noted that bumps reached a maximum in some areas during August (time lag of spring rains and snowmelt) and decreased slightly in the winter months (time lag of dry summer months).

Crandell (1958) and Hammerquist and Hoskins (1969) did not emphasize the effects of construction-induced rebound within the Pierre Shale to engineered structures. However, the U.S. Army Corps of Engineers (1981), in their study of the nearby Oahe Dam Project (8 mi north of Pierre, S.Dak.) observed that where exposed faults in the Pierre Shale are natural zones of weakness, unloading rebound was more noticeable and concentrated along the fault trace. They concluded that gouge and breccia along and near the fault are conduits for surface-water infiltration and ground-water flow, and that this water flow could initiate and maintain a rapid and continuing (up to 6 mo) rebound after construction. When wet, the gouge containing smectitic and illitic clays would expand and lose frictional strength, allowing easy rebound movement along fault surfaces. The U.S. Army Corps of Engineers (1981) also found that rebound due to excavation characteristically reached a maximum near the bottom of an excavation, decreased near the toe of the slopes, and became a negligible amount at the extremities of the slopes. During the excavation for the Oahe Dam Stilling Basin, Underwood and others (1964) found that rebound-induced vertical movements in bedrock at depths from 10 to 40 ft were smaller, slower, and less variable than the rebound-induced movements near or at the excavated surface. They also found that deep (10-40 ft) movements did not diminish as rapidly with time as those at or near the excavated surface.

In a study of the geotechnical properties of the Pierre Shale, Nichols and others (1986) determined that as overburden loads are removed from underlying rock masses, either by mass-wasting processes or by engineering excavations, these rocks are initially deformed by rapid elastic rebound followed by much slower time-dependent elastic and inelastic rebound. Inelastic, non-recoverable rebound causes significant relaxation of the rock fabric that allows penetration of external agents, such as meteoric water, which cause further relaxation. Rapid removal of overburden by natural or engineering excavation will result primarily in elastic rebound that initially masks the slower inelastic rebound that occurs with time and allows the rocks to slowly deform, relax, and deteriorate.

STRATIGRAPHIC NOMENCLATURE

The only exposed rock near the Highway 14 site belongs to members of the Upper Cretaceous Pierre Shale. These members include the Crow Creek, DeGrey, Verendrye, Virgin Creek, Mobridge, and the Elk Butte (Crandell, 1958). Of these, the Verendrye, Virgin Creek, and Mobridge Members have dominant exposure; the Virgin Creek Member is the most prevalent. At the present study area of site H1, the exposed rock is part of the Virgin Creek Member. Because the Virgin Creek forms the best exposures and contains easily identifiable bentonite beds that serve as good marker horizons, it is the key Pierre Shale unit for understanding deformation (Collins, 1987; Nichols and others, 1987). Pleistocene sand-and-gravel deposits, containing numerous Tertiary rock fragments, cap some hills near the study site (Crandell, 1958).

GENERAL FAULT CHARACTERIZATION AND AGE

The faults within the study area have as much as 105 ft of displacement, commonly have normal separation (hanging wall down relative to the foot wall without the true slip direction and true amount of movement known), and trend either northwest or northeast (Collins and others, 1988). As noted by Crandell (1958), the ubiquitous clay soil (gumbo) that covers the shale bedrock in central South Dakota hinders observation of faults except where they are exposed in natural or artificial cuts. Thus, it is difficult to get an accurate inventory of the faults throughout the study area without extensive subsurface investigations (e.g., Nichols and others, 1988). Where they are exposed, these faults can be traced from one stream bank across valley floors and onto the opposite bank. At some localities, part of a stream's orientation is controlled by faults that intersect the stream valley. For example, a fault described by Collins and others (1988) has a 66-ft-high, east-west-trending scarp that parallels a bend along Ash Creek. This fault scarp has caused the stream to flow parallel to the fault trace for a distance of 490 ft.

Fault gouge is commonly less than 0.2 in. thick but can be as much as 7 in. thick (Collins and others, 1988). Euhedral selenite crystals occur within the thicker (greater than 1 in.) fault gouge and along the fracture surfaces of the highly fractured or brecciated scarps. Fault breccia may extend as far as 3 ft on either side of a fault. Iron oxide coats the breccia and fracture surfaces. Well-developed mullion structures have been observed on highly fractured fault scarps. The mullions are as much as 6 ft apart and as long as 38 ft, the height of some scarps.

Near site H1, faults cutting the Virgin Creek Member are known to extend upward into the Mobridge Member (Upper Cretaceous); however, the faults have not been found to extend any further upward than into the Mobridge (Crandall, 1958). This suggests a Late Cretaceous or younger age for these faults and for the faults within our study area.

HIGHWAY DAMAGE AT SITE H1

For this paper, one site (site H1; fig. 1) about 18 mi west of Pierre, S. Dak., on a new section of U.S. Highway 14, was investigated where the highway has been damaged at its intersection with a fault. The geologic setting at this highway cut in the Pierre Shale is similar to that encountered at the Oahe Dam Site (Underwood and others, 1964; U.S. Army Corps of Engineers, 1981). At site H1, the highway pavement has been deforming since 1983 and continues to deform to the present (1989). This deformation manifests itself as a recurring west-facing bump or scarp in the pavement that requires scraping and repaving. The adjacent ground surface and a nearby fence have been vertically offset. The fault associated with this deformation has an apparent dip of 55° E., with a strike of N. 30° W., and shows east-side-up, reverse separation. The development of deformation at this site began less than 6 mo after construction of the new highway in the fall of 1983. From 1983 to present, the South Dakota Department of Transportation has performed three highway patches and two grindings to correct for the continuing fault damage at site H1 (J.D. Hammell, Geologist, S. Dak. Department of Transportation, oral commun., 1988 and 1989).

A fault scarp can be traced for at least 90 ft only within the excavated area on both sides of the road (fig. 2) at site H1. The maximum scarp height (1.6 ft) is found in the borrow ditches next to the road at the lowest point of road excavation. However, the height of the scarp decreases upslope and disappears into the cut slopes on either side of the highway. No sign of the fault scarp could be found in the fields immediately to the north and south of the highway cut.

From highway-construction data, we estimate that 54,650 short tons of material was removed along 200 linear ft above the block west of the fault. Less material (23,740 tons) was removed along 200 linear ft above the block east of the fault. Total tonnage of overburden removed was 78,390 tons, which represents an average of 25 vertical ft of material. Stratigraphic correlation of the Pierre Shale at site

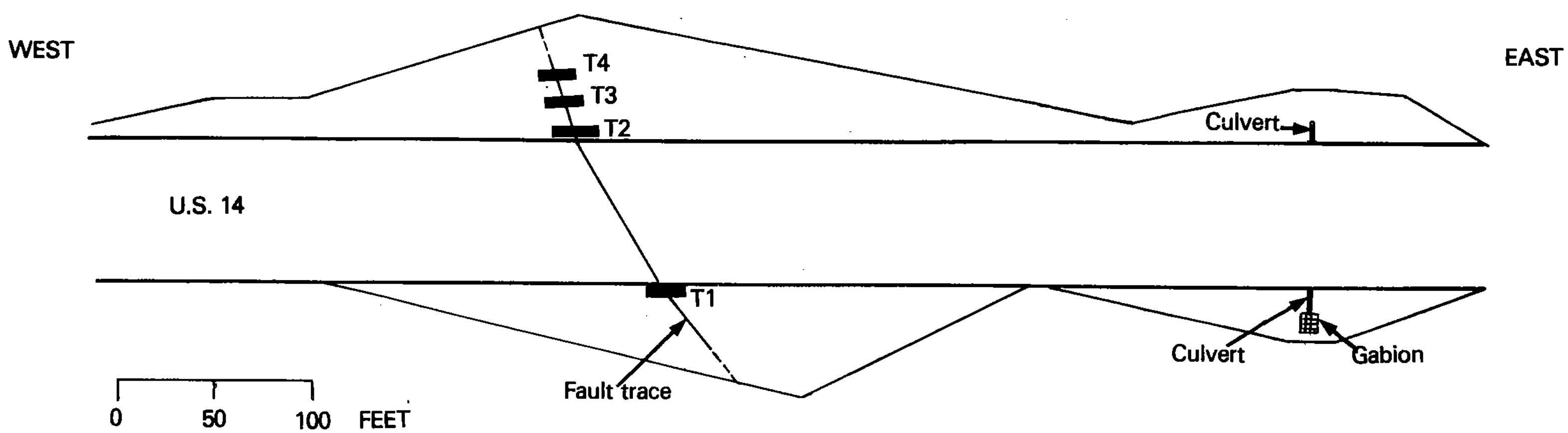

FIGURE 2. Schematic map view of site H1 in a cut along U.S. Highway 14 showing locations of trenches T1 through T4 intersecting a fault. The trend of the fault is N 36° W with an apparent dip of 55° E.

H1 to exposures in a tributary to Government Draw, immediately to the east, indicated that the site is located on rock within the upper part of the Virgin Creek Member of the Pierre Shale.

Nichols and others (1986) have determined that the state of stress for the Pierre Shale at a site about 15 mi to the west of site H1 is nearly lithostatic; that is, all principal stress components are nearly equal in magnitude. This may be a result of overconsolidation--a condition created from surface erosion of about 1,300 ft of overburden, causing horizontal stresses to be equal to or slightly greater than the vertical stress. As a result, upward-directed rebound may occur when overburden is removed; this rebound could reactivate fault zones near the new surface and cause damage to any structure built on it, such as a highway.

The pavement failure at site H1 has been attributed to the wetting and swelling of clays in the fault gouge (Vern Bump, Geotechnical Engineer, S.Dak. Department of Transportation, oral commun., 1986). The moisture required for wetting is derived from infiltrating meteoric water that flows along the fault zone and (or) associated fractures. To test this hypothesis, the exposed fault was trenched on both sides of the road to a depth of 5-6 ft in June 1988 (Collins and others, 1988). Trench T1 (27 ft long by about 2 ft wide) was excavated in the borrow ditch immediately south of the highway (fig. 2). Trench T2 (16 ft long by about 2 ft wide) was dug in the borrow ditch on the north side of the road. Trenches T3 and T4 (of similar length and width as trench T2) were dug north and up slope of the road. All four trenches were oriented east-west and intersected the fault.

Fault-gouge thickness was variable, but was usually 1-2 in., and did not exceed 3 in. in any of the trenches. The gouge did not appear to thicken upward. Although a few bentonite marker beds were found in the walls of the trenches, we were still unable to determine the exact amount of separation across the fault. The minimum observable separation in trench T1 was about 2 ft.

Trench dampness increased downslope from T4 to T1. For instance, 6 hrs after excavation, T1 had 2 in. of standing water in the bottom of the trench (fig. 3). We observed water flowing from the fractures and partings within the shale about 3 ft above the trench floor and possibly up through the floor. This latter source of water could not be verified due to the amount of debris and water covering the floor. Trench T2 also had about 2 in. of water covering a few feet of its length at the east end of the trench. The walls of T1 and T2 appeared to become wetter with depth. Trenches T3 and T4 did not have water on the floors and the walls were not as wet.

To better understand the physical properties of the shale and fault gouge at site H1, shale samples were collected across the fault near the base of trench T1 (fig. 4) to determine X-ray clay mineralogy, moisture content, natural bulk density, and Atterberg limits. Results of these analyses are presented in tables 1 and 2.

X-ray clay-mineralogy analysis (table 1) shows a lower percentage of expandable clay in the fault gouge than in the shales away from the fault. Shale samples A through F (fig. 4) are composed of a swelling clay that has a predominantly a monovalent exchange ion (for example, sodium and (or) potassium) and average 97 percent expandable clays (table 2), which is close to the expandability of pure smectite. Samples HG, X, Y, and Z (gouge samples), and nearby shale samples H and I (fig. 5) are indicative of a swelling clay that has dominantly a divalent exchange cation (for example, calcium and (or) magnesium). Although shale sample J has a 001 spacing indicative of the higher (97 percent) expandability group, its expandability percentage is similar to that of the gouge samples and shale samples H and I. Sample J probably represents an intermediate state of calcium ion exchange (illitization); the average expandability of this illitic clay is 77 percent. Shale sample G also has a monovalent exchange cation similar to samples A through F, but, unlike this group, G has a lower percentage of expandability (80 percent) and a smaller 001 spacing. These properties are characteristic of the divalent exchange cation group and indicate that sample G has undergone some illitization. Because gypsum was found in the gouge and on the shale-fracture surfaces, we suggest that the gouge clay has become more calcium-saturated by exposure to wetting and drying cycles of calcium-bearing waters. Therefore, upon exposure to wetting and drying cycles and in the presence of calcium-bearing minerals, the gouge becomes less expandable. This supports the findings of Hammerquist and Hoskins (1969) that calcium-rich gouge material is much less expandable than sodium/potassium-rich materials.

The moisture and natural-bulk-density values (table 2) indicate an increase in shale matrix void space away from the fault. This void space may be a result of relaxation of the shale matrix, due to the removal of overburden during highway construction, as proposed by Nichols and others (1986). The

FIGURE 3. View looking east along trench T1 in borrow ditch on south side of U.S. Highway 14. Length of trench is 27 ft. Height of north wall is 6 ft. Water depth in trench is as much as 2 in.

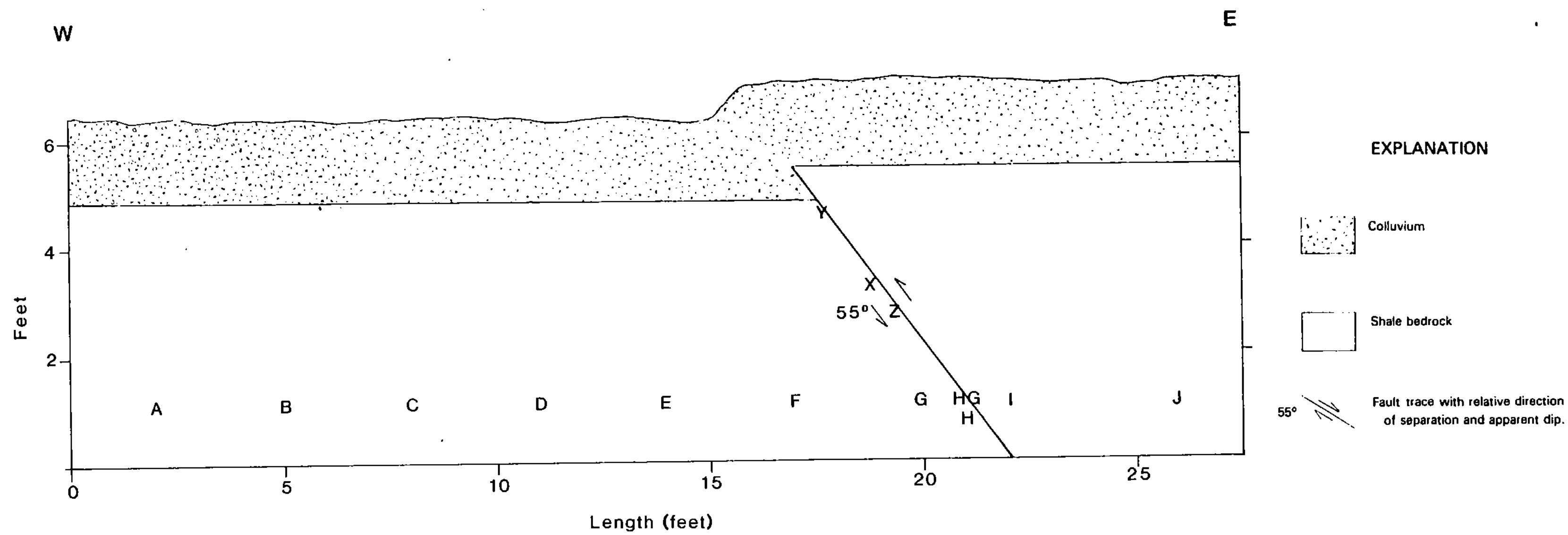

FIGURE 4. Schematic diagram of the north wall of trench T1 at site H1. Letters indicate positions of selected samples collected for analysis (tables 1 and 2).

TABLE 1

X-RAY DIFFRACTION ANALYSIS OF CLAYS FROM THE PIERRE SHALE

(Sample locations are identified in fig. 4)

Sample	Distance from fault zone (ft)	Spacing of basal plane (001) in Å (air-dried samples)	Percent expandability of clays (glycol-saturated)	Illite/Smectite to Illite ratio
X	0	15.0	87	3:1
Y	0	14.7	70	3:1
Z	0	14.6	82	3:1
HG	0	14.7	80	1:1
H	0	14.0	65	4:1
I	1	14.1	80	5:1
A	19	11.6	97	1:1
B	16	12.3	90	3:1
C	13	11.0	100	2:1
D	10	11.5	90	?
E	7	11.3	100	7:1
F	4	12.1	100	4:1
G	1	11.4	80	4:1
J	5	12.0	75	2:1

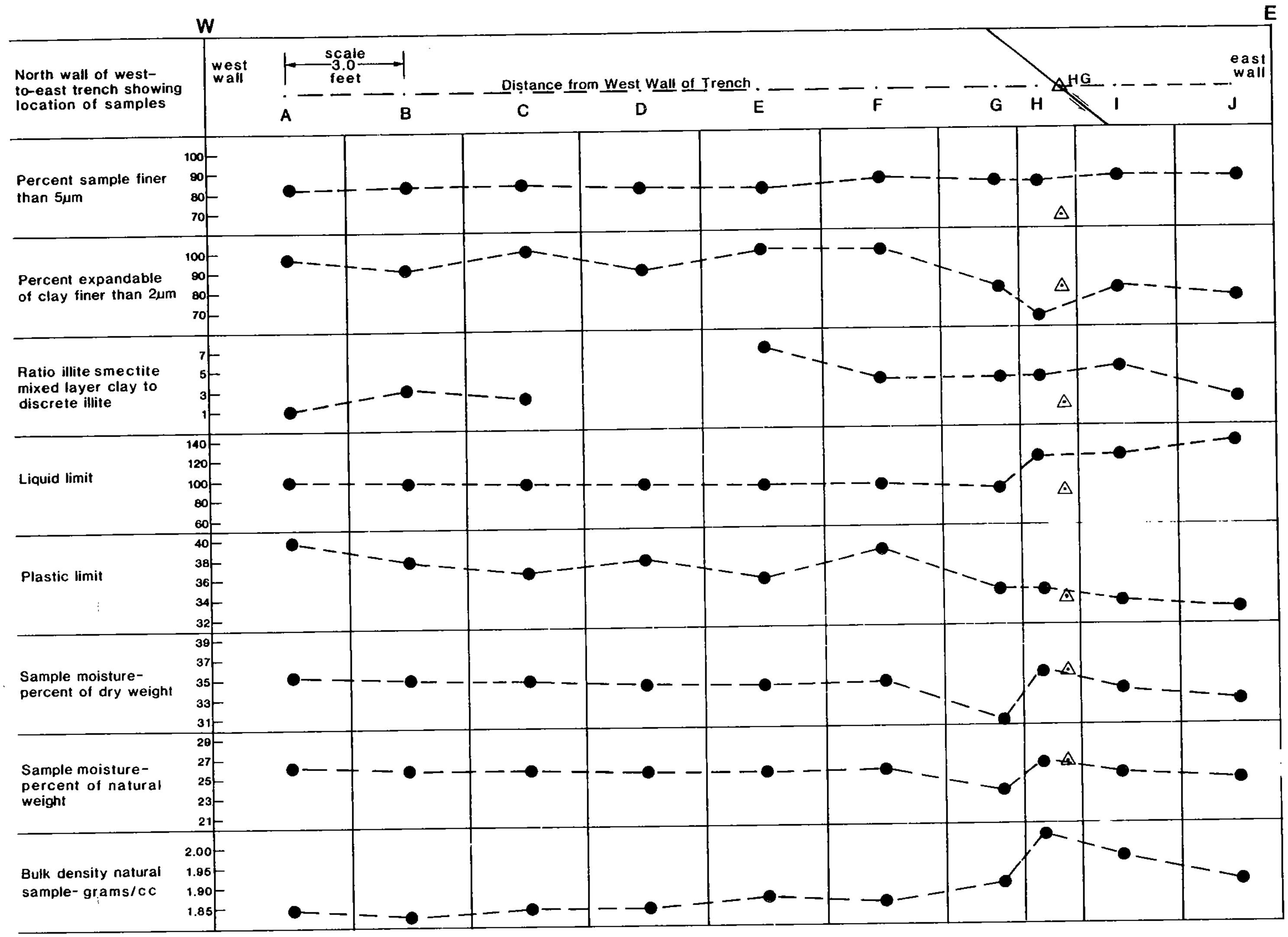

TABLE 2. Geotechnical data for selected Pierre Shale samples taken from the north wall of trench T1 at site H1 (fig. 2).

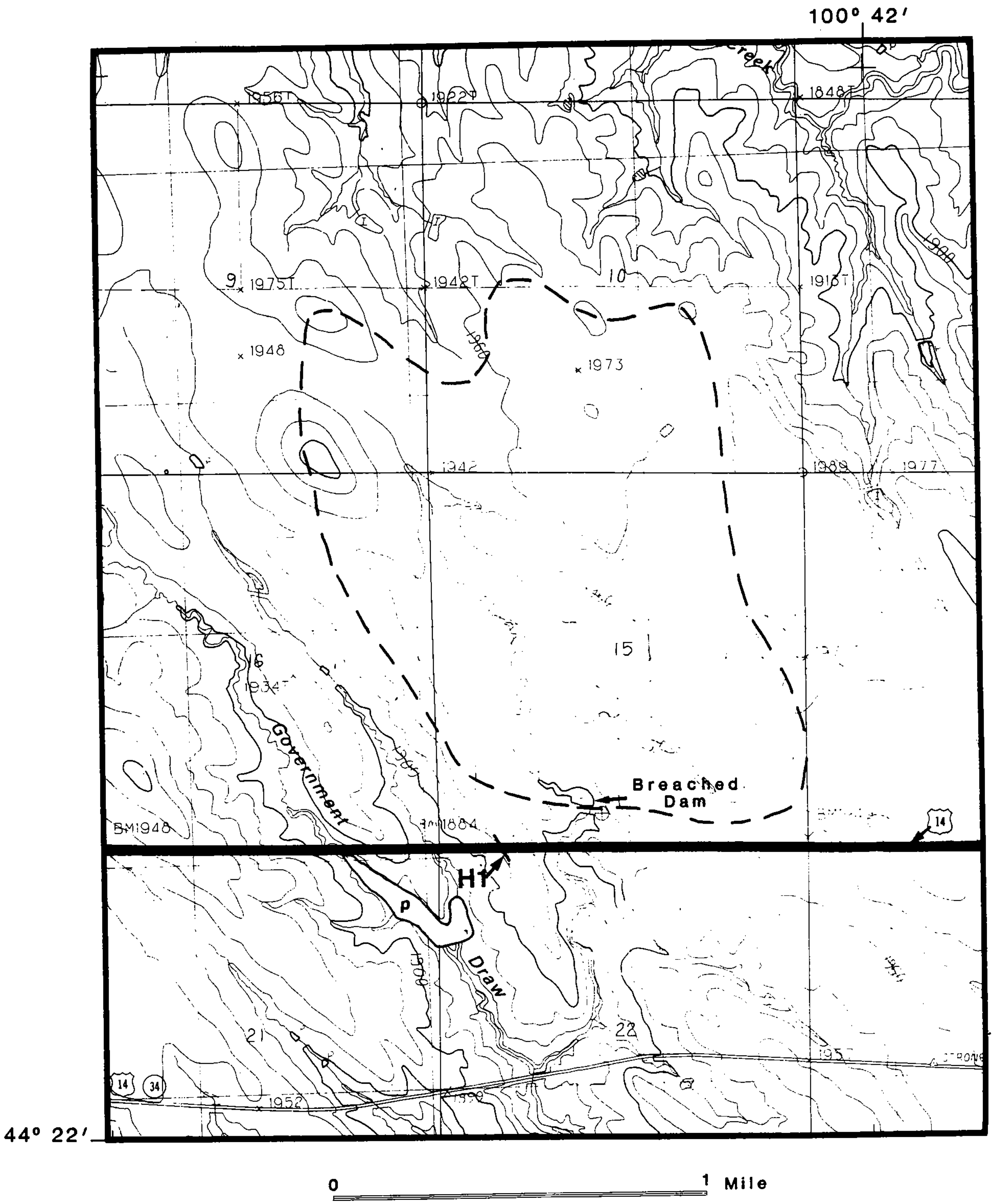

FIGURE 5. Index map showing the fault exposure at site H1 on new section of U.S. Highway 14 (solid line), the new (since 1982) stock dam (P), and the enlarged drainage area of the tributary to Government Draw (dashed line) developed above the breached dam (arrow). The culvert and gabion affected by upstream migration of a knickpoint in the tributary are shown in greater detail in figure 2.

higher densities found adjacent to the fault (samples G through J) could be a result of greater compaction and induration near the fault that prevented the shale matrix to relax as much. The slightly higher moisture content of the disaggregated-gouge sample (HG) is a result of having better permeability and a greater surface area so to better absorb water.

The gouge-size distribution shown in table 2 does not support the idea of Hammerquist and Hoskins (1969) that gouge weathers to a finer grained material than the original bedrock. In our study, analysis indicates that more of the finer-than-5-μm-size clay exists in the unaltered shale than in shale found in and near the fault. Perhaps mechanical compression at the fault surface provides enough energy for the finer clay particles to be compacted into larger particle sizes. However, we do not known why shale sample H has the lowest 2-μm-size content.

Although we did not note a decrease in fault-gouge thickness with depth, a decrease in fault-gouge thickness has been observed in cores obtained at other localities along Interstate 80 by Hammerquist and Hoskins (1969). They observed that, at depths of 10 ft or more, the gouge was usually less than 1 in. thick, but near the surface, less than 5 ft, the gouge zone was often thick (up to 6 in.). No explanation was offered for this upward thickening of gouge. However, the high content of swelling clays (30-100 percent) in the Pierre Shale (Nichols and others, 1986, app. 1) and exposure of fault zones to cyclic wetting and drying could explain the upward thickening of gouge. As a result of cyclic wetting and drying along the fault zone and nearby fractures, the shale fault-block walls and breccia should break down by both desiccation and water absorption. Then, during movement along the fault, this added weathered material adjacent to the fault gouge would develop slickensides. Eventually, this weathered shale material would not be distinguishable from true fault gouge. This effect would decrease with depth where shale would not be affected as much by drying. Thus, the cyclic wetting and drying would affect only the upper parts of the fault zone and would give the overall appearance of an upward-thickening gouge and breccia zone.

The permeability of the Pierre Shale matrix at a site 15 mi to the west of site H1 has been calculated to be $3x10^{-13}$ ft/s (Nichols and others, 1986), a value too small to allow rapid water migration through the shale matrix to accumulate on the floors of trenches T1 and T2 within a few hours. However, on the basis of the moisture differences in the trenches, as well as the precipitates (iron oxide and gypsum) on the shale-fracture surfaces and in the gouge, water migration is dominated by movement through the shale-fracture system. Water ponds on the downthrown side of the fault under the southern borrow ditch of the road. Although the fault is located on a hill, it is positioned in a general topographic low (Collins and others, 1988). Ground water flowing through the fractured bedrock from higher elevations may be contributing more water to the fault zone than is derived from the immediately adjacent hill(s). Collins and others (1988) also noted that a nearby stock pond in Government Draw and its tributary east of and lower than site H1 (fig. 5) remained full even during the severe summer drought of 1988. This is in contrast with other stock ponds within the study area that were very low or dry, which suggests that this pond on Government Draw is filled mainly by ground water traversing a fracture system rather than by surface runoff.

To determine if the fault at site H1 could be an extension of any nearby faults, we examined nearby Government Draw and its tributaries for faults of similar strike (Collins and others, 1988). Resulting field data showed that faults found in a tributary east of site H1 did not align with the highway fault. However, investigation of Government Draw and its tributary immediately west of highway fault H1 was hampered by the lack of bedrock exposures and the presence of a filled stock pond.

REGIONAL DEFORMATION

Several studies were undertaken to investigate the possibility of recent regional tectonic deformation in the study area as a cause of fault movement along a segment of U.S. Highway 14. First, the most recent leveling data obtained from NOAA (National Oceanic and Atmospheric Administration) were examined by Collins and others (1988) for significant elevation changes. First-order level lines located in the eastern part of the study area were measured on June 21 to September 27, 1949, and on June 18 to August 8, 1951. Comparison of these data to precipitation records for the same time periods showed that all elevation increases along the level line could be attributed to swelling of clays in the wetter months (Collins and others, 1988).

Second, a fluvial geomorphic study of stream length and slope, tributary stream order, and changes in sinuosity within the area indicated recent stream rejuvenation that may be due to either glacial rebound or other tectonic causes (Jones-Cecil and others, 1988).

Third, knickpoints in streams are sometimes initiated by recent faulting, as was the case following the Hebgen Lake earthquake in 1959 (Morisawa, 1962). We investigated a knickpoint observed in a tributary (fig. 5) to Government Draw just east of site H1 and south of U.S. Highway 14 (Collins and others, 1988). At this location, a bedrock knickpoint had migrated more than 217 ft upstream within a 10-month period during the 1986-87 season. Below this knickpoint the downstream valley was deepened by 9 ft and widened to over 12 ft. When re-visited during the 1988 field season, it had migrated another 82 ft upstream. The presence of highly fractured bedrock where faults cross the valley may have accelerated migration of this knickpoint. No additional evidence for recent deformation-related knickpoint migration or initiation along the full length of Government Draw, nearby washes, and the remainder of this tributary was found during the 1985-86 or the 1988 field season. Then we discovered that a stock dam north of U.S. Highway 14 (SW¼ sec. 15, T. 5 N., R. 28 W., fig. 5) had been breached sometime after the 1977 field check for the 1982 update of the Lacy 7.5-minute quadrangle; the breach may have occurred as late as the major spring flood of 1984. The breaching of this dam increased the drainage basin area of this tributary by about 1.3 mi^2 above the knickpoint, and resulted in an increased volume of runoff that may have caused a rapid upstream migration of the knickpoint along a relatively short stretch of the tributary.

Fourth, localized basement faulting may be occurring in the southern part of the study area, as evidenced by a 4.2-magnitude earthquake that occurred at lat. 44.250° N., long. 100.724° W., in 1961 at a calculated depth of 14 mi (Gordon, 1988). However, no surface disturbances from this earthquake have been found.

Among these studies, only the earthquake and geomorphic data suggest possible tectonic activity in the region. Because the fault at site H1 was discovered only after the highway cut was completed, we suggest that regional tectonism in the form of glacial rebound or basement activity has not contributed to recent movement along this fault.

DISCUSSION

The evidence presented suggests the following scenario for the formation and maintenance of the highway bump at site H1. Excavation and the removal of approximately 37,000 yd^3 of soil and rock in the roadcut exposed a fault that had not been observed prior to road construction. The removal of about 25 ft of overburden triggered the rebound process along the near-surface extent of the fault and dilated fractures along and adjacent to the fault zone allowing surface water from above and ground water from below to infiltrate the fault zone and bedrock. In subsequent cycles of wetting and drying, the fault-zone gouge, composed initially of a high percentage of expandable clays, swelled. A simultaneous reduction in friction along the fault surface allowed the rebound process to continue at a slowly decreasing rate and contributed to additional mechanical disintegration and displacement along the fault zone. Chemical alteration of the expandable clays in the fault gouge by cation exchange was initiated and maintained in the presence of calcium-rich waters, resulting in the stabilization of the fault zone itself. Swelling of clay particles in the soils and fractured rocks adjacent to the fault zone continued and contributed to the growth of the highway bump in spite of corrective actions by road crews in their effort to mitigate the effects of the bump on road traffic.

SUMMARY AND CONCLUSIONS

Highway damage over faults in South Dakota has been explained by past workers as a result of either (1) compressional stress by highway use (Crandell, 1958), (2) differential uplift due to gouge swelling (Hammerquist and Hoskins, 1969), or (3) excavation-induced rebound (Underwood and others, 1964). From this study, we believe that initial road damage at the highway study site is a result of construction-induced rebound that has caused reactivation of the fault exposed by construction. Subsequent growth of the highway bump is driven by swelling clays in the fault gouge and adjacent soil and fractured bedrock. We do not believe that the entire fault is reactivated, but, rather, that relative movement occurred along only the upper (near-surface) part of the fault. This response to construction excavation is based on findings of the U.S. Army Corps of Engineers (1981) made during and after the excavation of the Oahe Dam Stilling Basin in the Pierre Shale. To mitigate rebound-induced fault movement and bump formation at site H1, a drainage system placed in the south borrow ditch parallel to the highway and sloping away from the road should allow rapid draining of surface water away from the weathered, fractured shale and fault breccia zone. This should minimize the wetting and drying cycle effects and prevent further ponding under the highway, thus reducing disintegration of the shale into a swelling clay-size fraction that would facilitate rebound movement by reduction of friction along the fault surface. D.D. Eberl (Geologist, U.S. Geological Survey, oral commun., 1988) recommended adding a gypsum layer under the highway. This layer would provide calcium ions to exchange with the sodium ions of the swelling clays of the shale, gouge, and breccia to reduce clay expansion and perhaps disintegration due to cyclic wetting and drying. Another consideration is to allow the rebound to stabilize after excavation and then to fill in the downside of the fault scarp, rather than to relevel the road grade by excavation. Further excavation would only create additional instability that could result in renewed rebound, in new accessible pathways for water encroachment, and further swelling of expandable clays.

To generalize about similar highway deformation in South Dakota based on this study of only one deformation site is not practical. A number of questions still remain with regard to site H1; they include 1) why the fault has reverse rather than normal separation movement, 2) how does moisture affect the displacement of the fault and deformation of the roadway? and 3) how much overburden is needed to be removed to induce rebound. The answers to these questions may focus on the combined mechanisms involving fault rebound and swelling clays and should be investigated further.

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SEISMIC RECORD VERSUS GEOLOGIC RECORD IN THE SOUTHERN RIO GRANDE RIFT REGION

by

Jamie R. Barnes

Sergent, Hauskins and Beckwith
El Paso, Texas

ABSTRACT

At present, the El Paso-Las Cruces border region is considered to be in a zone of relatively low potential for significant earthquake activity. Only ten historic earthquakes of significant nature have had epicenters in the region.

This record of historic seismicity, however, does not fully represent the apparently active Quaternary history of faulting in south-central New Mexico, northern Chihuahua and far west Texas.

Recent investigations southeast of El Paso have establilshed the presence of significant Quaternary activity on the Amargosa fault, a previously uncharacterized major fault about 70 km south of El Paso. Evidence of at least one Holocene and several Quaternary surface rupturing events are displayed by the fault.

These investigations, coupled with recent works by Machette, Seager, Gile and others indicate normal fault activity as recently as 1000 years BP along the Organ and Franklin Mountain Ranges and on intrabasin faults in the Tularosa Basin and Hueco Bolson.

The apparent disparity between the historic seismic record and the Quaternary geologic record should be addressed in any regional evaluation of seismic risk.

INTRODUCTION

The southern Rio Grande rift region including related structural grabens such as the Salt Basin Graben (Figure 1), is an area of known Quaternary and Holocene fault activity. The area, however, has a relatively small historic seismic record. As a result, various authors have viewed seismic risk in the region differently.

Richter (1959), published a regional seismic risk map which included the study area in a zone in which major damage from earthquakes could occasionally occur (Figure 2). In contrast to Richters map which characterzed the Rio Grande Rift in a zone of relatively high seismic risk (occasional Modified Mercalli Intensities (MMI) of IX), the current Uniform Building Code, based largely on maps produced by Algermissen (1969), has the area located in Seismic Zone 1. Seismic Zone 1 corresponds to MMI of V to VI.

Recent studies, some of which are detailed below, suggest that the southern Rio Grande rift region may have a more significant potential for earthquake activity than previously suspected. Numerous faults have been identified which have experienced repeated Quaternary displacement, some as recently as 1,000 years ago.

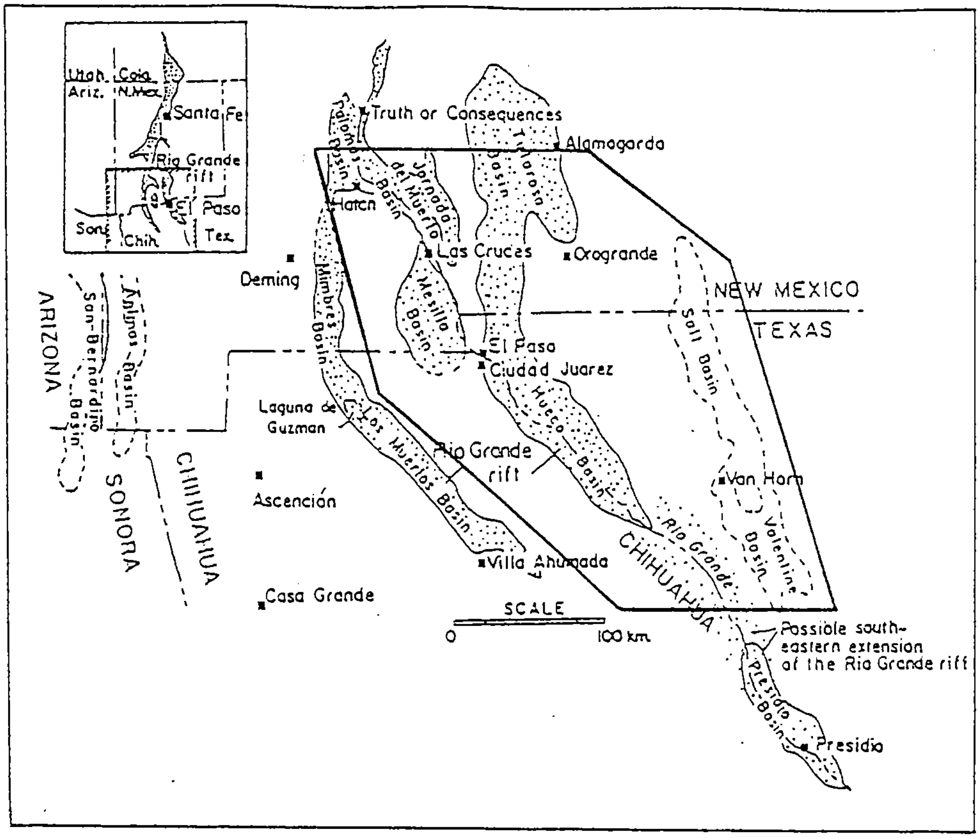

FIG. 1-Location of study area, modified from Seager and Morgan, 1979.

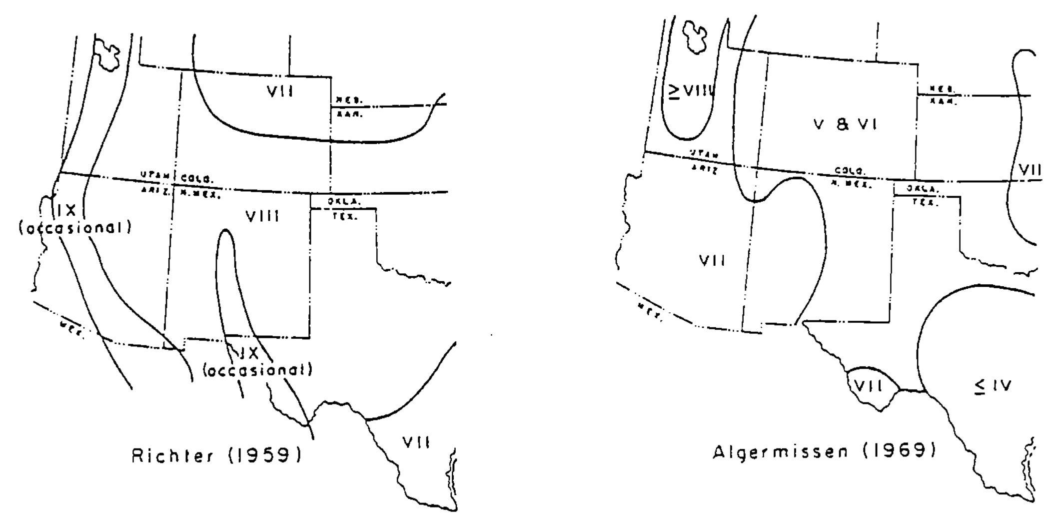

FIG. 2-Seismic risk maps. Intensities according to Modified Mercalli Intensity Scale of 1931.

REGIONAL SEISMICITY

The seismicity of west Texas and southern New Mexico is discussed by Dumas et. al., (1980), Dumas (1981), and is included in earthquake summaries of the state of Texas by Davis et. al, (1985, 1989) and of southern New Mexico by Sanford and Toppozada (1974) and Northrup (1976). Seismicity in the entire Basin and Range province and the Rio Grande rift are detailed in many compilations, including those of Sanford et. al., (1972, 1979), Smith (1978) and Askew and Algermissen (1983).

Records of earthquake activity in the region date back to the 1850's. Data used in earthquake compilations relied primarily on "felt" reports prior to 1962 (Northrup, 1976; Davis et.al., 1989). Since 1962 the majority of earthquake epicenters and magnitudes have been instrumentally derived.

Data prior to the 1960's, in the form of felt shocks, have been assigned intensity values based on the 1931 Modified Mercalli Scale.

Earthquake magnitude is a measure of the intrinsic size of an event, whereas intensity values are a representation of perceived earthquake effects. These effects are dependent on the distance from epicenters and are generally related to ground acceleration (Davis et.al., 1989). Magnitudes and intensities can not be directly correlated. Magnitudes, where assigned, are generally estimations from reported intensity data as interpreted by general correlations of intensity and magnitude by Richter (1958), Slemmons, et.al., (1965) and Wiegel, (1970).

Intensity values are strongly influenced by population density within the region affected by the shocks. Since much of the study area has had a relatively sparse population, it is not unlikely that even moderate shocks may have gone unreported. In addition, reported shocks may have been significantly lower in intensity than the actual earthquake strengths (Sanford et.al., 1972).

The reliability of early earthquake reports should be carefully considered. Inaccurate and exaggerated reports of shocks are common, and many other shocks can be attributed to nontectonic sources such as sonic booms and explosions (Davis et.al., 1989). Even with the inherent innaccuracies, the pre-instrumental record is of considerable value since it's length of record greatly exceeds any instrumental record of seismicity.

The historic record, as indicated by epicenter locations in Figure 3, is dominated by a moderate to low level of seismicity within the Rio Grande rift system in southern New Mexico and by moderate to low earthquake activity in a dispersed pattern throughout Trans-Pecos Texas and in adjoining Chihuahua, Mexico.

Ten historic earthquakes (exceeding M=2.7), have had epicenters located near El Paso. These events include an earthquake in March of 1923 with a maximum MMI of VI and two MMI VI events in May, 1969 (Davis et.al., 1989). These events are believed to have occurred near faults associated with the Rio Grande rift (Muehlberger et.al., 1978; Davis et.al., 1989). Sanford and Toppozada (1974), list eleven felt earthquakes prior to 1961 and six instrumentally detected quakes in southeastern New Mexico and west Texas between 1961 and 1972.

In a compilation of earthquake activity in New Mexico, Northrup (1976), state that 76 percent of all recorded New Mexico earthquakes were within the Rio Grande rift zone, but that of these earthquake, 96 percent were restricted to the area between Socorro and Albuquerque. This implies that less than 4 percent of recorded events are known to have occured south of Socorro. Sanford et.al. points out that this record may be somewhat biased by population distribution. However, even taking population in to account, the record seems sparse. Northrup (1976), furthur stated that between 1947 and 1967 only five earthquakes in three different years were recorded between Socorro and the Texas-New Mexican border.

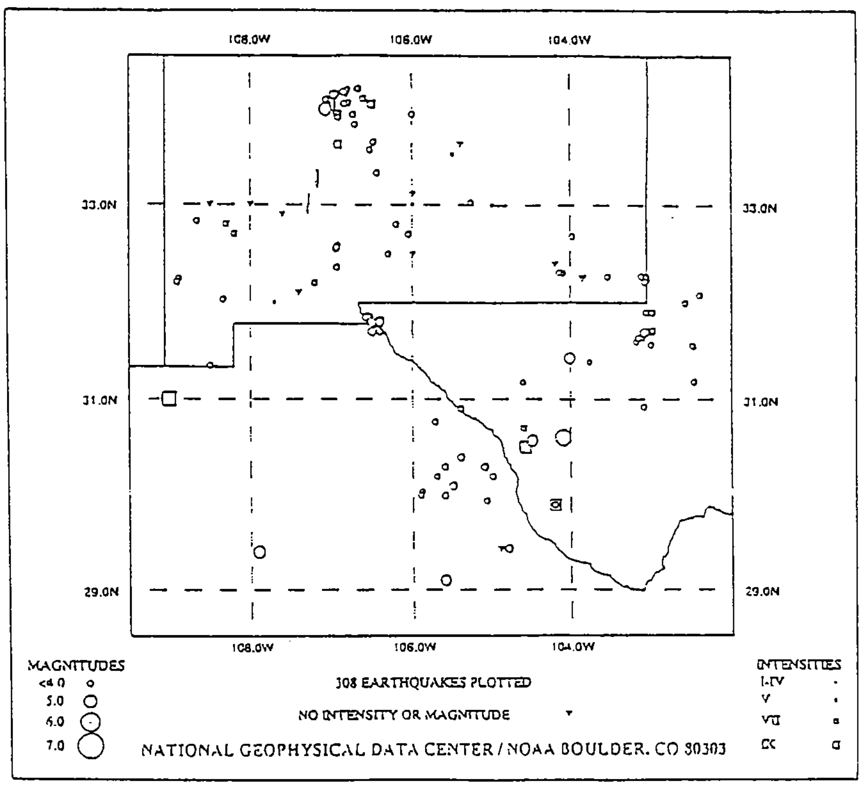

FIG. 3-Earthquake epicenters in west Texas, southeastern New Mexico, and north-eastern Chihuahua.

Instrumental data on earthquakes indicate few shocks M>2.7 occuring in the region between Las Cruces and Socorro (Sanford et.al., 1972). Shocks felt were attributed primarily to activity south of Las Cruces in a possible extension of the Rio Grande rift (Sanford et.al., 1979).

In the southern portion of the study area several earthquakes have occured in the Valentine, Texas area, including the M=6.4 August 1931 Valentine earthquake (Doser, 1987). This is the largest historical earthquake in the region.

Recent studies by Dumas (1980) and Davis et.al. (1989) in the Marfa Basin and Van Horn area, have recorded numerous microearthquakes. These events have been related to movement along portions of the Rim Rock fault, other faults in the Salt Basin Graben and unmapped faults in the Marfa Basin. The microseismic record indicates at least the southern portion of the study area may be more seismically active than previously known.

3. REGIONAL GEOLOGIC SETTING

The study area is located within the Rio Grande rift portion of the Basin and Range Tectonic province. As a result of a complex regional tectonic history, basement rock types, strain histories and resulting structural control on Late Cenozoic features are heterogeneous and complex.

Tectonic development of the Basin and Range province included two main periods of activity (Zoback and others, 1981; Price and Henry, 1986). The earlier phase of pre-middle Miocene (pre-10 to 13 m.y.) extension was characterized by west-southwest extension, locally high strain rates, listric normal faulting, and shallow basin development. The later phase is marked by west-northwest extension, possibly lower strain rates, high-angle normal faulting and the formation of grabens and half-grabens. Recent Basin and Range style deformation is indicated by widespread late Pleistocene and Holocene fault scarps (Muehlberger and others, 1978; and historical seismicity (Doser, 1987).

The structural setting of the region largely reflects Late Cenozoic Basin and range style extension. Many workers have suggested that there is substantial geological and geophysical evidence for southeastward extension of the active Rio Grande rift system into westernmost Texas (e.g. Seager and Morgan, 1979).

The northern portion of the study area includes the classic north-trending enechelon grabens of the Rio Grande rift which extends from central Colorado to the Texas-New Mexico border region.

The southern portion of the study area is generally considered to represent an eastern extension of the Basin and Range Province, however, several authors (Seager and Morgan, 1979; Muehlberger, 1978; Keller, 1985) suggest the region may be a southeast extension of the Rio Grande rift zone, which varies somewhat in volcanism, heat flow and fault trends from those that characterize the classical rift zone. This extension of the Rio Grande rift zone to the southeastern margin of the Basin and Range includes the Salt Basin and Marfa Basin and portions of northern Chihuahua. For the purposes of this report, the study area will include these basins but does not imply the acceptance of a similar structural setting.

The main structural features of the study area are elongate horst mountain blocks and subparallel grabens, including the Mesilla and Hueco-Tularosa Bolsons and the Salt Basin. These basins are structural grabens controlled by prominent frontal faults on their margins with more distributed fault occasionally occuring opposite major frontal fault margins (Seager, 1980, 1983; Goetz, 1980). Quaternary faulting and the potential for large earthquakes in this region have been recognized by many workers, including (Goetz, 1980, Muehlberger and others, 1978, Muehlberger, 1978, Thenhaus and Wentworth, 1982, Nakata and others, 1982, Machette, 1987 and Doser, 1987). Muehlberger and others (1978) delineated two north-northwest trending zones of Quaternary normal faults in the Trans-Pecos region. The eastern zone extends for 300 km from southern New Mexico through the Salt Basin graben to Van Horn, Texas. The western zone continues south from the Rio Grande Rift in southern New Mexico and follows the Tularosa and Hueco Bolsons to El Paso. It then turns southeast along the Hueco Bolson, south of El Paso and includes the Amargosa fault.

Figure 4 shows prominent Quaternary faults including the Organ fault, the Amargosa fault, the East Franklin Mountains fault the Salt Basin fault zone and the Mayfield fault. Other Quaternary fault zones may exist that have not yet been recognized.

Studies on specific faults in the region indicate an abundance of major faults with obvious Quaternary movement. A detailed discussion of a few of these faults are presented below:

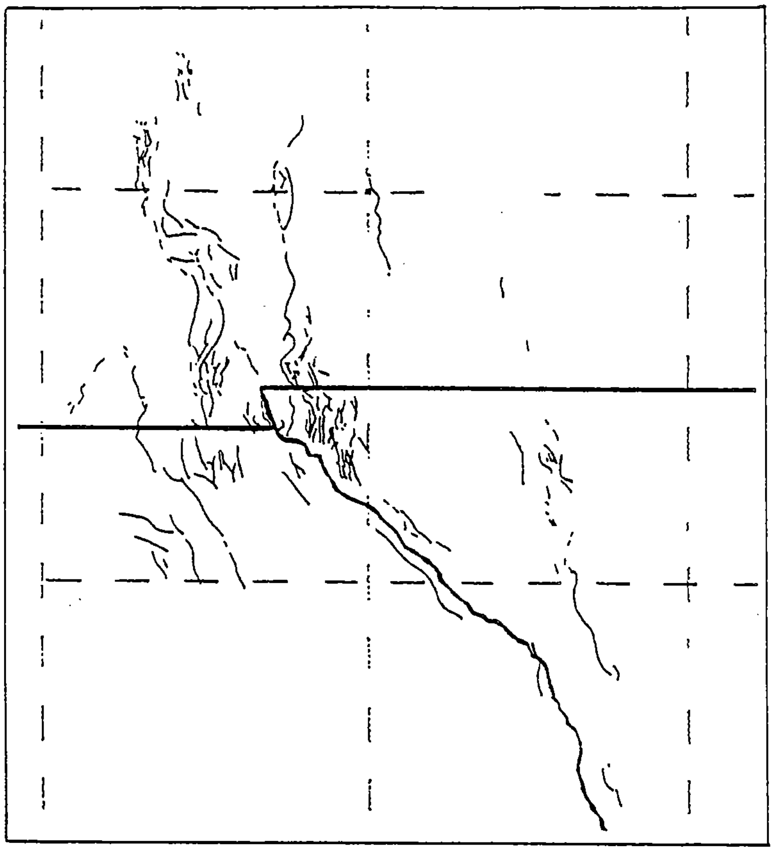

FIG. 4-Map showing Quaternary faults in the study area, summarized from Machette, 1987; Muehlberger, 1978; Seager and Morgan, 1979; Henry and Price, 1985 and Seager, 1980.

ORGAN MOUNTAINS FAULT

The Organ Mountains fault, part of the longer San Andres, Organ and East Franklin Mountains fault system, (Seager, 1981), has the youngest reported evidence of major fault activity in the study area. The 42 km long normal fault, which borders the eastern margin of the Organ Mountains, is characterized by a series of prominent fault scarps (Machette, 1987). Scarps heights range from less than 1m to about 37m (Beehner, 1990).

Pedologic work by Gile (1987) and scarp degradation studies by Machette (1987) indicate latest movement along the fault at 5,000 years or less and perhaps as recent as 1,000 years BP. Machette, (1987), stated that scarp-morphology data indicates two major faulting events in the Holocene. Scarp heights of 2 to 4.6m have been interpreted as the amount of displacement occuring during single Holocene rupturing events along the Organ Mountains fault (Beehner, 1990; Machette, 1987).

EAST FRANKLIN MOUNTAINS FAULT

A distinctive fault that bounds the northwest margin of the Hueco Bolson is the East Franklin Mountains fault (Machette, 1987); or the East Boundary fault (Lovejoy, 1971, 1972). The fault extends a length of about 45km along the eastern margin of the Franklin Mountains (Machette, 1987). Single event scarp heights typically range from 1.7 to 5 meters with an overall displacement of Quaternary deposits as evidenced in scarp heights typically ranging from 9 to 60m, but may be as much as 125m (Sayre & Livingston, 1945). Machette, 1987 states that evidence obtained from morphologic, pedologic and geologic data indicates that the East Franklin Mountain fault has had recurrent Quaternary movement and has probably been active in the Holocene. Rupturing events may have occurred as recent as the last

4-5,000 years (Seager, 1983). Associated with the prominent range front faults of the Organ-East Franklin Mountains are a complex zone of north-trending Quaternary normal faults about 30 km wide and more than 60 km long, (Henry and Price, 1985). The details of faulting have not been carefully assessed, however, Seager (1983) considers faults of this zone to be younger than 0.5 m.y., based on an offset soil developed on bolson sediments. According to Seager (1980, 1983), faults within the Hueco Bolson and along its eastern boundary are genetically related to the prominent frontal fault system along the Franklin-Organ uplift on the west side of the Hueco Bolson, which controls the geometry of the west-tilted Hueco graben.

Individual faults within the Hueco Bolson have lengths of about 5 to 30 km (Seager, 1983). This zone of faults form a series of horsts, grabens and tilted blocks within the basin sediments (Seager, 1980). The extent to which seismic shaking would be associated with movement on these relatively minor intrabasin faults is unclear.

AMARGOSA FAULT

Along the northeast flank of the Sierra San Ignacio-Sierra la Amargosa in Chihuahua, Mexico, is the Amargosa fault. The Amargosa fault is a spectacular range-front fault on the southwestern edge of the Hueco Bolson (Keaton, 1989). Preliminary analysis of stereoscopic low-sun-angle photography and several days of field reconnaissance have revealed a prominent, very continuous fault scarp (class I tectonic front). The presence of very steep scarp slopes (up to 27°) and scarp heights of between 2.8 and 32 meters suggests several rupture events of Quaternary surfaces. Limited field evidence (Barnes, 1989, Keaton, 1989) suggests at least one, and possibly several, Holocene surface-rupturing events have occured along the fault. The linear north 40-50° W striking fault scarp has an end-to-end length of at least 47 mi (75 km), (Keaton, 1989; Barnes, 1989). The linearity and continuity of the fault scarp, and the greater prominence of the central part of the scarp, suggest that the Amargosa fault probably is not segmented. Where seen in the field, the Amargosa fault dips 65-75° to the northeast, and has predominantly normal displacements. Graben-like extensional features along the fault may represent lateral components associated with the fault although these relationships are vague (Barnes, 1989).

SALT BASIN FAULT ZONE

Quaternary faulting in the Salt Basin graben is discussed by Goetz (1977, 1980) and Muehlberger and others (1978). Quaternary and Holocene fault scarps are prominently displayed on the western margin of the Salt Basin and locally on the eastern margin and within the basin. Frontal faults on the west side of the Salt Basin graben occur as a segmented zone with individual fault segments as long as 30 to 34 mi (48 to 55 km). Goetz, 1977, measured Quaternary displacements of 1 to 3 meters in faults along the western graben margin. A brief reconnaissance in the area by the author revealed very distinct fault scarps attesting to the apparent recency of movement.

The Salt Basin fault zone is part of a 260-mile (440-km) north-northwest-trending zone of faults and related structures that forms the eastern limit of the Basin and Range province (Goetz, 1985).

MAYFIELD FAULT

Extending for more than 80 km along the east side of the Van Horn Mountains and Sierra Vieja, the Mayfield fault forms a prominent semi-continuous fault scarp. The fault is composed of three major portions, each with a slightly different character. The northern portion trends about N10°E, terminating into three subparrallel splays with a throw of about 1m (Muehlberger, et al., 1978) down to the east on each splay. The middle section forms a single 35 km scarp striking northwest with an average scarp height of 2 m (Muehlberger, et. al, 1978). The southern portion of the Mayfield fault trends north to northeast with surface displacements of up to 7 meters (Muehlberger, et. al, 1978). Muehlberger and others (1978) showed that the Mayfield fault locally exhibits evidence of recurrent Quaternary displacement. The Mayfield fault is believed to be a potential source of the M = 6.4 1931 Valentine earthquake (Doser, 1987). A variety of authors suggest alternate faults in the vicinity as the source of the Valentine events.

4. DISCUSSION

This paper does not intend to imply an analysis of seismic risk. It does, however, strive to point out an apparent discrepancy between the geologic and seismic record of the region.

The faults presented above, along with numerous other apparently active Quaternary faults in the regions, show anple evidence for major seismic shaking in the recent geologic past. Although some historic earthquake activity can be attributed to these faults, the lack of major earth shaking events appears to be at odds with the geologic record. Estimated magnitudes on major faults based on length and displacement parameters, coupled with historic data, indicate a potential maximum magnitude event of 6.4 to 7.5 (Beehner, 1990; Krinsky, 1986; Doser, 1987), thereby producing major ground neutrons.

Despite the obvious geologic evidence of recent tectonic movement in the southern Rio Grande rift, (Sanford et. al, 1979) states that seismic activity in the region was no greater than in neighboring physiographic provinces for the years 1962-1972.

The absence of seismicity on many recognized Quaternary faults may reflect recurrence intervals longer than the historical record (approximately 150 years), rather than lack of potential for future damaging earthquakes. Seismicity in the Basin and Range province tends to be temporarily clustered and in general does not always correlate with mapped Quaternary faults (Smith, 1978). Earthquake recurrence intervals for major surface rupturing earthquakes in the Basin and Range province are typically about 1,000 to 100,000 years. Similar rates are expected in the study area. Machette, 1987, indicated that recurrance intervals on the Organ Mountain fault were typically about 4,000-5,000 years. Such a low recurrance interval rate combined with a relatively short historical record may lead to an inadequate characterization of seismic potential. This is not to say that geologic evidence implies that seismic activity will occur along any of the fault zones mentioned. Logic, however, dictates, that given an apparently high level of activity within the immediate geologic past, it is reasonable to assume that the potential exists for at least some activity to occur in the near future.

CONCLUSIONS

The Southern Rio Grande rift region is characterized by a moderate to low level of historical earthquake activity that is inconsistent with the regions's Quaternary and late Quaternary faulting history.

The historical earthquake record in Southern Rio Grande rift region indicates some recent seismic activity. It does not appear, however, to provide an adequate representation of regional seismic activity or seismic risk. Quaternary and late Quaternary fault scarps in the area clearly indicate the existence of moderate to large magnitude surface rupturing earthquakes in the recent past.

While several recently active faults have been identified, the detailed fault parameters necessary to adequately characterize the seismic potential of the region are generally not available. Difficulties in characterizing seismic sources in the region include burial by young sediments, cover by recent eolian deposits, as of yet unrecognized fault systems, complex shatter patterns that have not been mapped in detail, and the lack of techniques to accurately assess the age of faulting events. Overcoming these difficulties and developing a better understanding of the relationship between short term seismic data, the geologic record and the long term assessment of geologic hazards will be an essential part of design parameters for critical facilities in the future. Future research efforts should focus on the relationship between regional seismicity and potential seismic sources. Of particular interest, will be detailed investigations on recurrance intervals and the timing of last faulting events.

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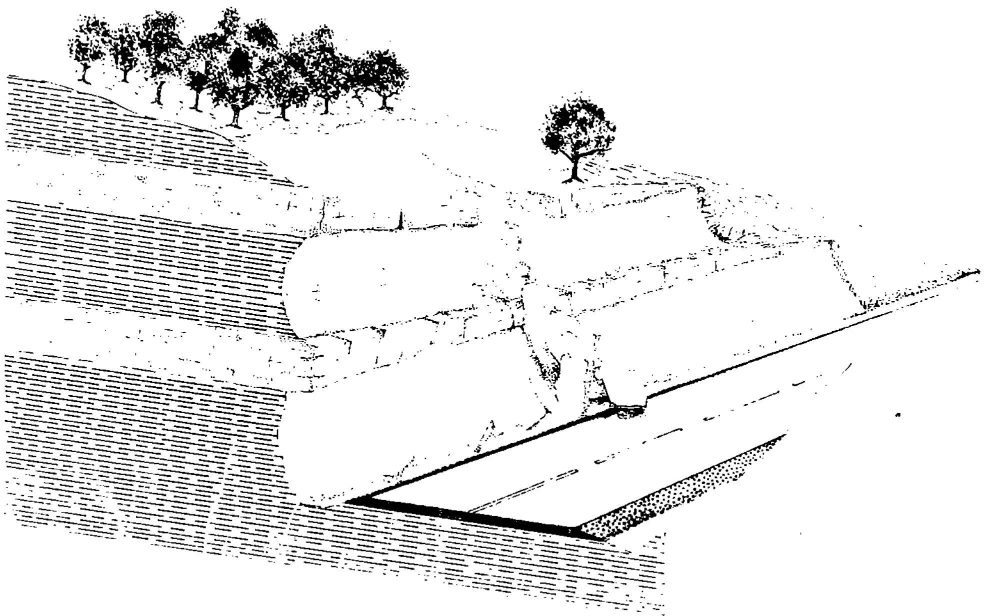
ROCKFALL MITIGATION ALONG I-40,

COCKE AND CUMBERLAND COUNTIES, TENNESSEE



BY
HARRY MOORE
GEOTECHNICAL SECTION
TENNESSEE DEPARTMENT OF
TRANSPORTATION

PREPARED FOR PRESENTATION AT THE 41 ST ANNUAL HIGHWAY GEOLOGY SYMPOSIUM, ALBUQUERQUE, NEW MEXICO, AUGUST, 1990

"Rockfall Mitigation Along I-40, Cocke and Cumberland Counties, Tennessee "

By
Harry Moore
Tennessee Dept. of Transportation
PO Box 58,Knoxville, Tn. 37901

ABSTRACT

Over the past five years the Tennessee Department of Transportation has been involved in a number of rock fall mitigation projects requiring innovative remedial attention. Two of these projects are located along Interstate 40 in Cocke and Cumberland Counties,Tennessee.

In Cocke County (near the North Carolina State Line) Interstate 40 is located in the mountainous Blue Ridge Province - which is underlain by faulted and fractured Precambrian metasediments. This stretch of I-40 has been prone to numerous rock slope wedge failures and dangerous rock fall events. Rock fall mitigation measures employed on this section of Interstate 40 included scaling and trimming, wire meshing, catchment fences and rock fall control fences.

Geologic conditions along Interstate 40 in Cumberland County consist of Pennsylvanian age flat-lying strata which are located atop the Cumberland Plateau Province. Differential weathering of the alternating shale and sandstone units have resulted in precipitous cut slopes and numerous rock fall events. A rock fall related highway fatality spurred a temporary rockslope scaling project which resulted in a number of innovative rock slope scaling concepts. A follow-up and permanent rock fall remedial project was undertaken the following summer. This project involved the use of cut slope layback and an alignment shift/depressed catchment ditch combination.

These innovative rock fall mitigation projects along Interstate 40 have resulted in the application of effective rock fall remedial techniques leading to geotechnically safer transportation routes.

INTRODUCTION

In recent years the Tennessee Department of Transportation has undertaken major remedial projects with the intent of reducing the occurrence of injurious rockfall events along highways in East Tennessee. Two of these projects involved heavily traveled sections of Interstate 40 where rock fall events have forced closure of the highway, caused thousands of dollars in damage, and caused injury and death to the motoring public.

This paper will attempt to document two of the more innovative rock fall remedial projects. The dissemination of this information is felt necessary in order to continue to improve the development of the science of Engineering Geology.

Although the two case studies chosen are dissimilar in location, geology, and remedial treatment, it is the wide range of treatment experiences that provides the uniqueness that demands documentation. As we earth scientists continue to study and understand how "civilization" can interact successfully with mother earth, it is imperative that we share with each other our successes as well as our failures. The premise of this paper is indeed to share the results of engineering geology experiences relating to rock fall problems and their cures.

Location

The two chosen case studies are located in two non-adjacent physiographic provinces of East Tennessee: The Blue Ridge and the Cumberland Plateau (Figures 1 & 2). The Ridge and Valley province separates the two areas. As one might surmise the geology and topography are also quite different in both provinces, requiring each area to be treated with different and unique geotechnical concepts. Accordingly the genesis of the rock fall problem and the resultant remedial concepts are also dissimilar. It is this difference of problem and solution that provides the range of experiences which were chosen to be detailed in this paper.

The first case study area (Case Study A) is in Cocke County where I-40 traverses the Blue Ridge Mountains in extreme eastern Tennessee. Situated along the eastern border of the Great Smoky Mountain National Park, this project area lies along the last three miles (4.8 km) of I-40 in Tennessee (from Hartford, TN to N. Carolina State Line) where the Pigeon River cuts a gorge through the Blue Ridge Mountains.

This area is underlain by folded and faulted precambrian age metasediments of the Ocoee Supergroup.

LOCATION OF REPORT AREA
ALONG I-40
COCKE AND CUMBERLAND COUNTIES
TENNESSEE

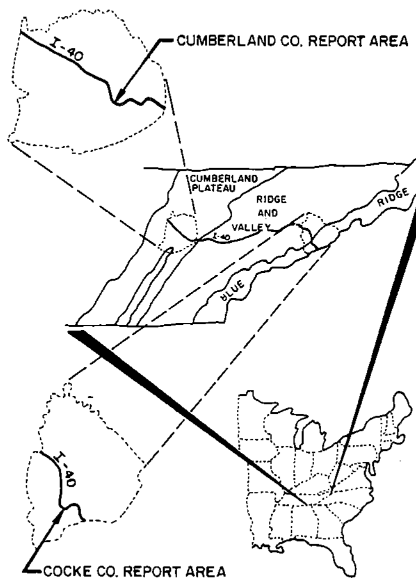


Figure 1. Location of Report Area.

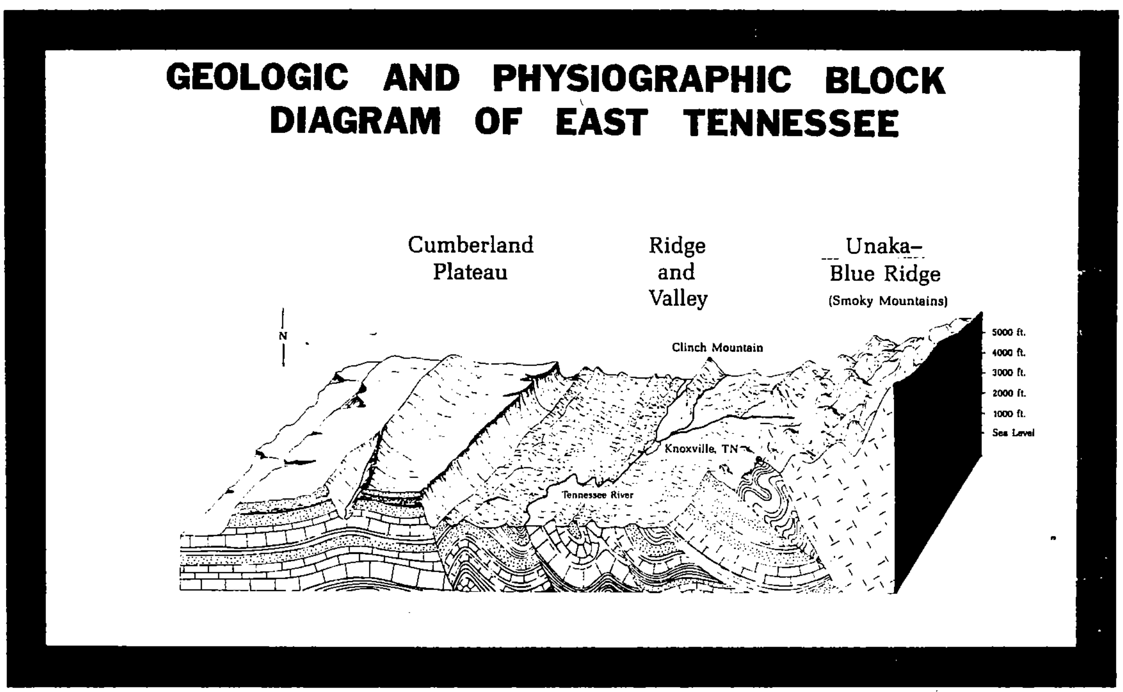


Figure 2. The general geologic structure and surface features of the project areas is shown on this block diagram of East Tennessee (Moore, 1988).

The location of the second case study area (Case Study B) is in Cumberland County where I-40 traverses a topographically high area of the Cumberland Plateau section of the Appalachian Plateaus Province. This project area involves the section of I-40 that was constructed across the Cumberland Mountains just east of the small town of Crab Orchard, Tennessee.

CASE STUDY A

Case Study Area A involves an area underlain by folded and faulted precambrian age metasediments. These metasediment strata are severely broken and contain numerous planes of fracture cleavage. Combined with the dipping structure and cleavage fracture, these rocks contain the kinematics for rock slope failure as well as rock fall events (Figure 3).

Highway construction through this area of the Blue Ridge during the 60's resulted in numerous jagged and precipitous rock cut slopes. These cut slopes have intersected and exposed the fracture and bedding planes of the rock strata which has resulted in numerous rock fall events and wedge failure landslides.

During the 70's and early 80's the rock fall events progressively worsened resulting in numerous episodes of rock fall. The rock fall events ranged from single boulders to 7000 cubic yard (5352 cubic meters) wedge failure landslides which closed portions of I-40 for up to 2 weeks at a time (Figure 4).

As a result of worsening rockslope stability and repeated safety concerns associated with the rock fall hazard, an investigation into the rock fall problem was undertaken. A period of extensive and detailed geologic field mapping and surveying was initiated in the early 1980's.

It was decided to undertake a comprehensive rockslope remedial project to greatly improve the rock fall and landslide situation along the subject section of I-40 in Cocke County. The remedial concepts chosen to be applied to the subject rock slopes included scaling and trimming, rock bolting, wire meshing, catchment fences, and horizontal drains (Moore, 1986). In addition, oriented pre-splitting was used to remove large scale rock wedge overhangs (Moore, 1988). As a result of the study numerous areas of cut slope instability were identified and targeted for additional rock slope stability analysis. Cut slopes prone to rock fall events were documented from maintenance records and visual inspection.

This paper addresses only the concepts used to control the subject rock fall problem and not landslides of a large scale. These concepts include stabilization and protection methods and involve scaling and trimming, wire meshing, and catchment fences (Figure 5).

The stabilization methods used to control rock fall consisted mainly of scaling and trimming. The scaling methods included hand scaling with pry bars and mechanical scaling with crane suspended devices and pneumatic hammers.

The hand scaling required workers to rappel down cut slopes prying boulders and hand tossing loose debris and rock rubble to the roadway shoulder (Figure 6). All cut slopes were mechanically scaled prior to the hand scaling "clean-up".

Mechanical devices involved steel-toothed dozer tracks and metal plates hoisted by cranes (Figure 7). The tracks and toothed steel plates were pulled over the face of each rock cut; in essence "rubbing" or "sanding" the rock surface rendering it devoid of loose, broken rock (Figure 8). In some instances crane booms in excess of 200 feet (61 meters) were used to scale the higher cut slopes.

An additional mechanical device that received limited use on this project was the pneumatic hammer (which was mounted on a small crawler). The device was referred to as a "spider" due to its numerous hydraulic legs which enables it to traverse some of the flatter but jagged rock slopes. The pneumatic hammer was used to "chisel" off overhangs and loose boulders.

Some of the protection devices consisted of catchment fences (along the roadway) draped wire meshing, and rock fall control fences.

Upon completion of the scaling and trimming measures, the precipitous slopes were then covered with draped wire meshing in an effort to minimize any future rock fall from reaching the roadway surface (Figure 9). The draped wire meshing differs from the standard wire meshing in that the mesh material is affixed to a cable network which is anchored along the top of the cut slope. The mesh is then allowed to hang freely against the cut slope.

The purpose of using the draped meshing was to allow some rock fall to occur but guiding the falling rocks to the ditchline where they can be picked up by maintenance crews. In some instances where original

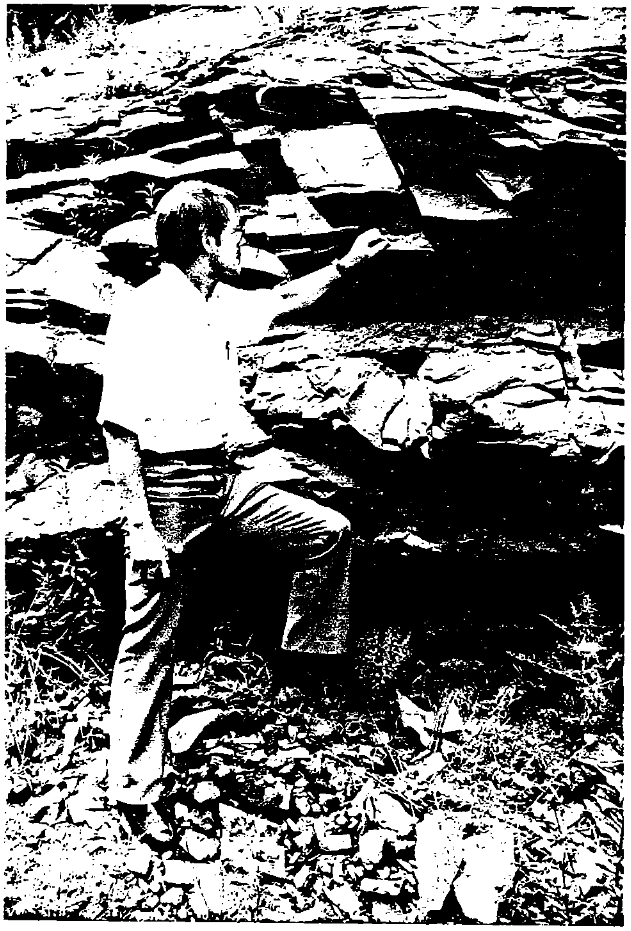


Figure 3. Joints, fracture cleavage, and bedding are discontinuities that contribute to the rock fall events along I-40.

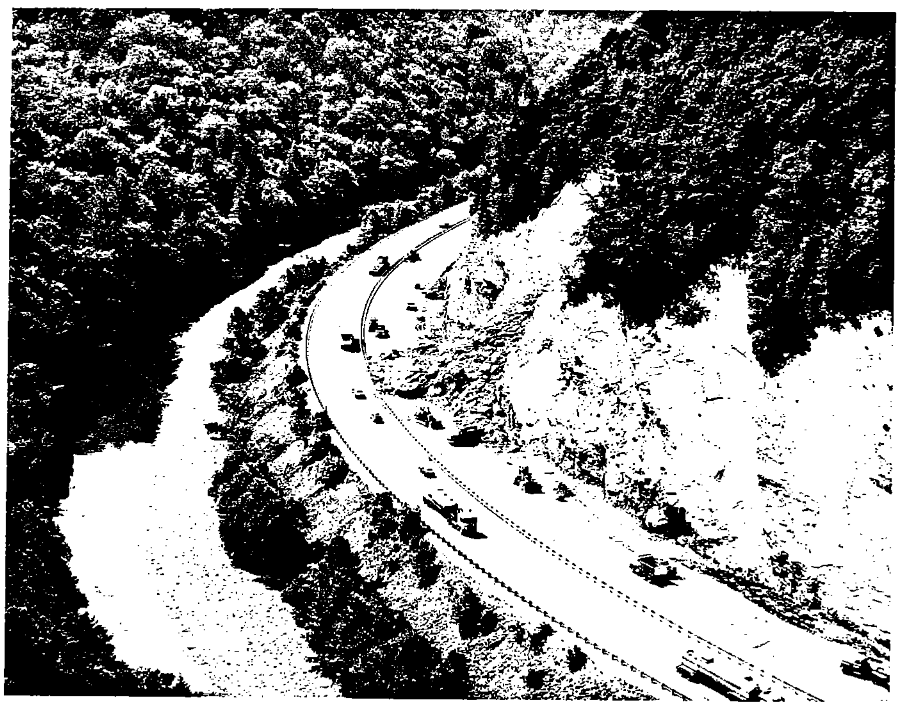


Figure 4. Rockfalls and landslides have closed sections of Interstate 40 in Cocke County, Tennessee; landslide occurred July 1976 (photo by George Hornal).

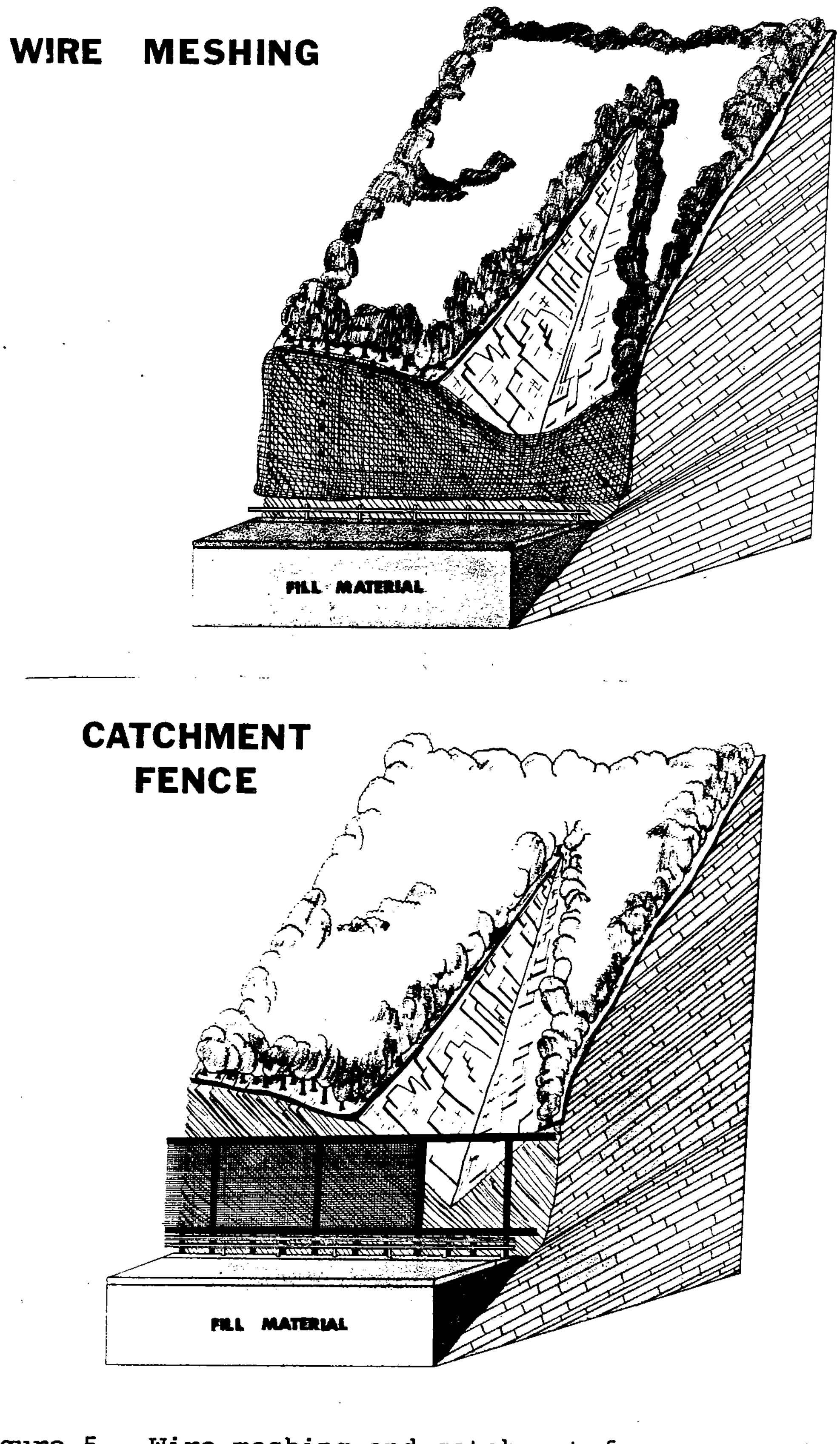


Figure 5. Wire meshing and catchment fences were two types of protection devices employed in the rockfall remedial project.

Figure 8. Scaling rock slopes along I-40 in Cocke County amounted to "sanding" the rock face of loose debris.

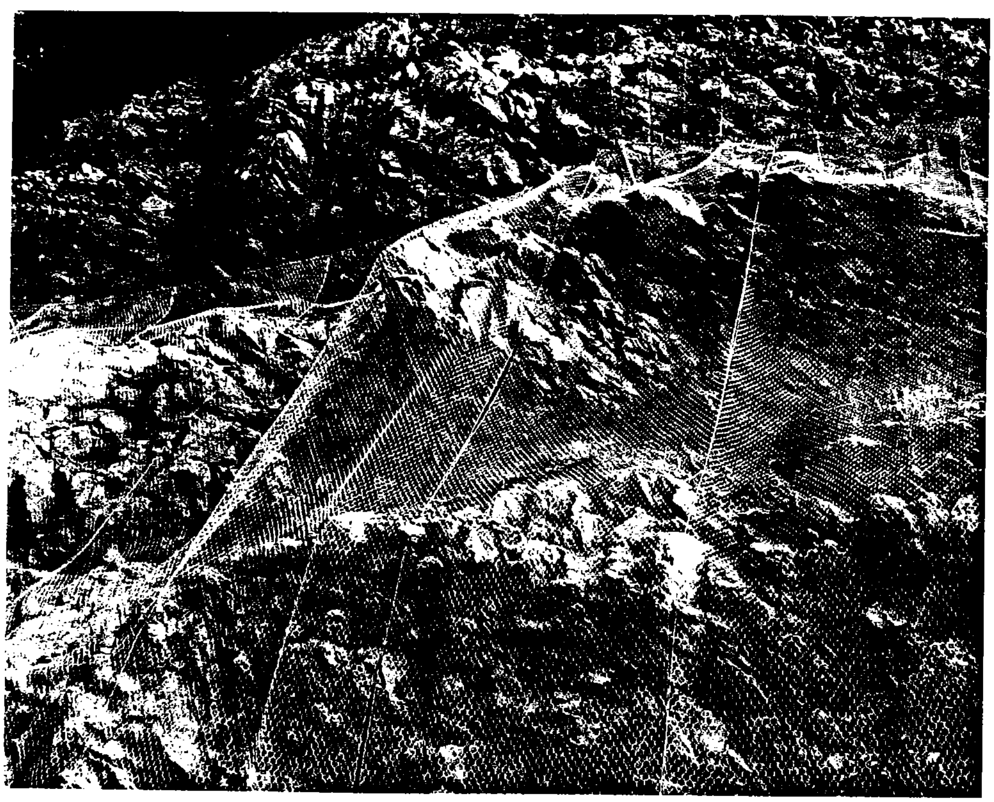


Figure 9. Draped wire meshing was used extensively along sections of I-40 in Cocke County.

construction resulted in multiple in-slope benches, a series of special rock fall control fences were used.

These special rock fall control fences were constructed along the outside edge of the cut slope bench. The rock fall control fence consists of a cable system supported by anchored posts and wire mesh. The next lower succeeding draped wire mesh is affixed to the cable. The cable is connected to a series of anchored steel posts about 5 feet (1.5 meters) in height.

The arrangement of the 5 foot (1.5 meters) high fence along the outer edge of the bench allows rock material falling onto the bench to be caught by the fence. Once the fence blocks the rocks, then the material is controlled in its fall behind the lower draped meshing (Figure 10). Unique in its design and employment, this concept has performed adequately since construction.

A special draped catchment and rock fall control fence was designed and constructed for the unique V-shaped landslide scars (wedge failure type landslides) which characterize numerous rock slopes in this area (Figure 11). These special catchment fences consist mainly of an anchored cable that stretches across the opening of a V-shaped wedge failure scar on a cut slope. The wire meshing is attached to this cable and allowed to hang freely over the face of the V-shaped opening. Rocks falling or rolling down the V-shaped slope are allowed to hit the free-hanging wire mesh, which dissipates the energy, keeps the rocks from reaching the highway, and allows the rock debris to drop down to the roadway maintenance ditch.

Catchment fences along the roadway shoulder were used to provide additional safety along the base of higher and more precipitous rock slopes (Figure 12). These catchment fences consist of a series of steel I-beams anchored into the rock with concrete. The I-beams are located along the edge of the roadway shoulder and are faced (front & back) with heavy gauge double twist wire mesh. Spacers were placed between the two layers of wire mesh in order to provide for energy dissipation.

Due to the close proximity of the catchment fence to the roadway shoulder, a "New Jersey" type barrier was constructed along the base of the fence facing the traffic lanes. This provided not only a safety measure for the moving traffic but also increased the restraining capabilities of the catchment fence.

Figure 10. Rockfall control fences were constructed along the outer edge of in-slope benches

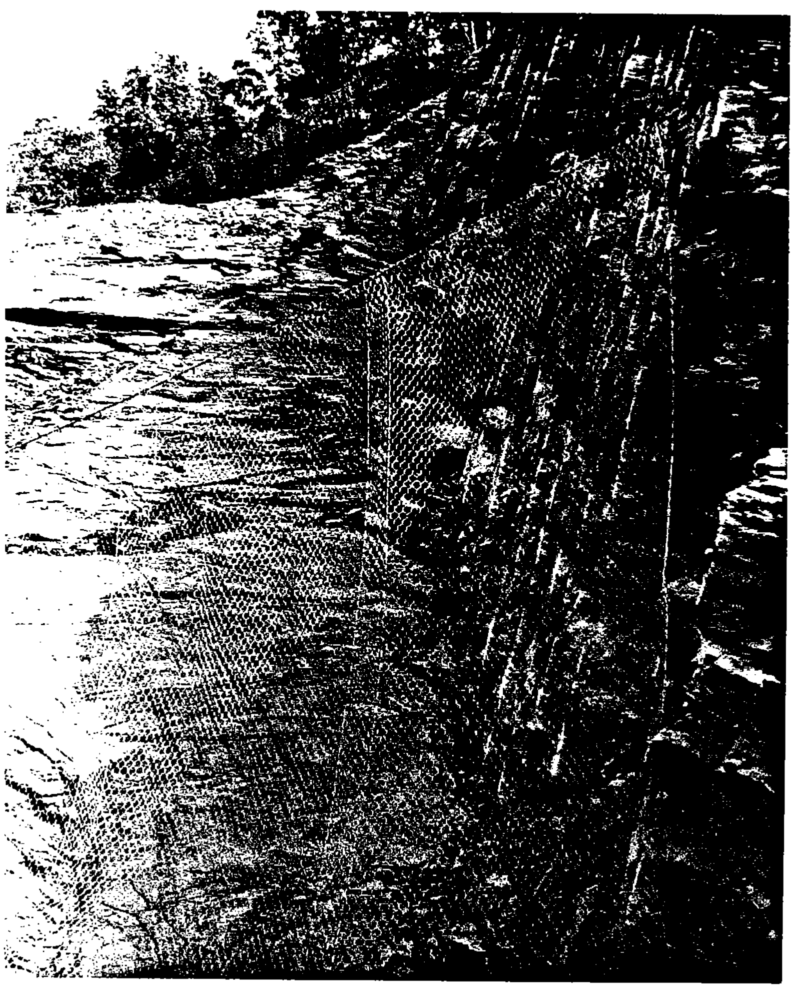


Figure 11. A draped catchment and rock control fence was constructed across wedge failure "V" shaped scars.

Figure 6. Hand scaling along I-40 in Cocke County helps to remove possible rockfall debris.

Figure 7. This 4'x5' (1.2x1.5m) metal plate was used in scaling rock slopes.

The rock slope remedial project took 2 working seasons to complete with an overall total cost of $7,786,012.36. Approximately 23.3% of this ($1,811,849.50) is the cost for direct rock fall remediation as previously described and detailed in this narrative. The cost for rock bolts, horizontal drains, special oriented pre-split construction and repaving are included in the overall total cost.

<u>CASE STUDY B</u>

The area of Case Study B is approximately 1.5 miles (2.4 km) in length and involves both the east and west bound lanes and the divided median of Interstate 40 near Crab Orchard in Cumberland County. Several instances of property damage and eventually a fatality resulting from the rock fall in this area prompted the implementation of a rock fall remedial project.

Case Study B involves an area underlain by flat lying and folded and faulted Pennsylvanian Age sedimentary rocks (Figure 13). These rocks are very fractured and contain thick bedded shales alternating with sandstone units 20 to 30 feet (6 to 9 meters) thick. Differential weathering allows the shale material to be eroded at a faster rate than the overlying sandstone layers resulting in large blocks of sandstone which overhang overhang the lower slopes. Rockfall problems arise when these sandstone blocks break loose along joints and fall or roll onto the highway (Figure 14).

The initial phase of the remedial project involved a temporary rockslope stabilization concept which was later followed by a more comprehensive and permanent repair. The total time required to complete the project was 2 years (beginning in January of 1987 and ending in December of 1988). The total cost was $5,704,824.19.

The project area consists of numerous vertical cut slopes in the 30 to 50 foot (9 to 14 meters) range and one large multiple benched cut slope in excess of 300 feet (91 meters) in height. All the cut slopes produced rock fall. The large 300′ (91 meters) high cut slope was very precipitous and produced the most damage and injury-related rock fall events. It was at this large cut that a fatality was recorded in December of 1986.

The temporary remedial project involved only emergency trimming and scaling of the subject rock

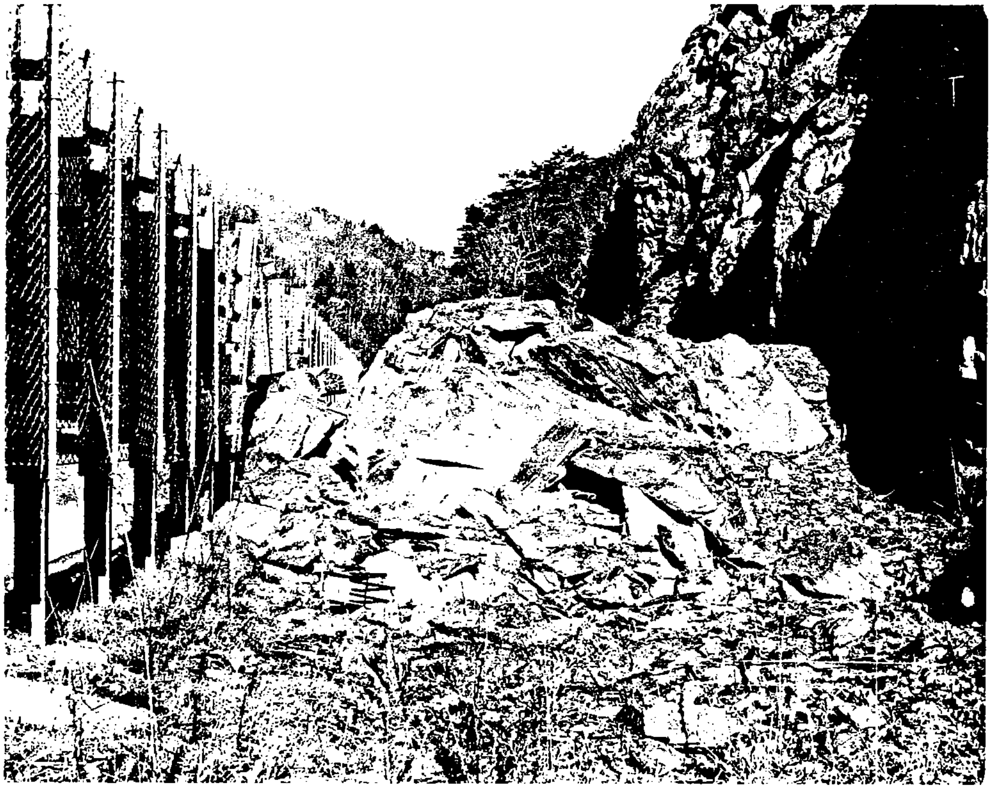


Figure 12. Rockfall catchment fences have proven to be beneficial as constructed along I-40 in Cocke County.

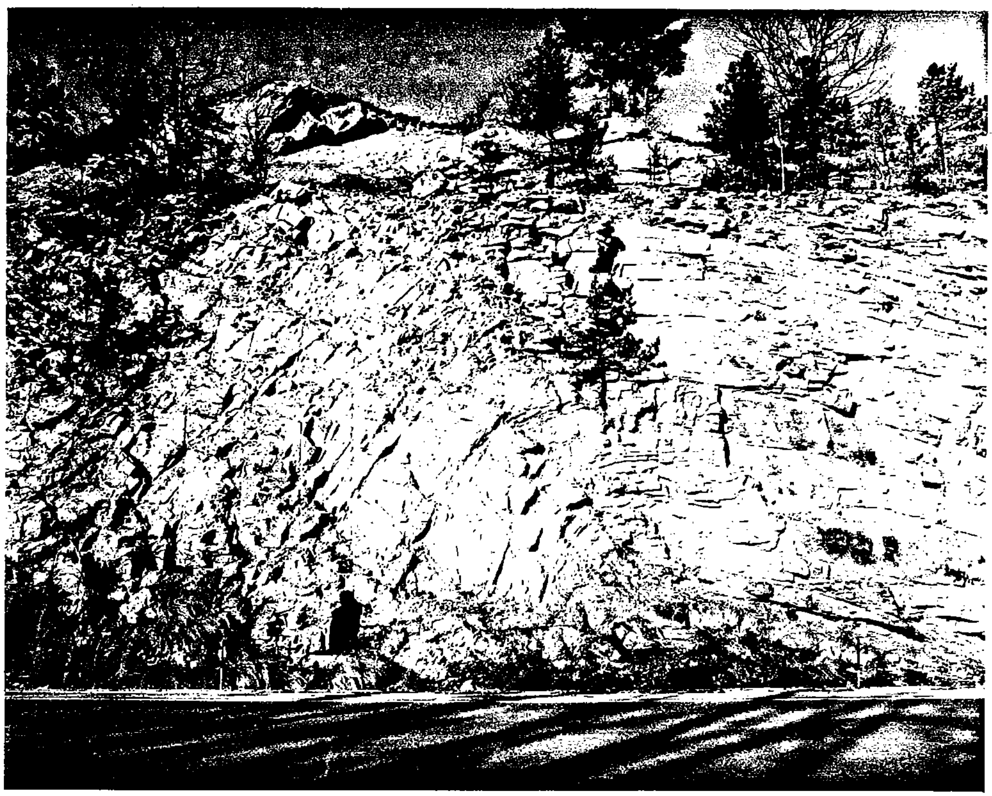


Figure 13. These folded Pennsylvanian Age sandstone strata reflect the structure found at Case Study B in Cumberland County.

Figure 14. Rockfall events along I-40 in Cumberland County have caused injury and property damage.

Figure 15. A dozer track was used to rub and pry loose rock from the existing cut face.

slopes. This was undertaken in order to "safe up" the cut slopes and to provide that the final repair contract be executed under traffic (without an immediate threat of damaging rock fall). The temporary project was completed within 2 1/2 months and involved the trimming and scaling of every cut slope along the 1 1/2 mile (2.4 km) section of interstate highway.

Scaling was accomplished by numerous methods which included the use of large "back-hoe" equipment, crane drawn dozer tracks, a crane hoisted "head-ache" ball, bulldozers wenched down the face of cut slopes, steel toothed dozer tracks wenched back and forth across the cut face, and finally hand scaling (Figure 15, 16, 17, 18). Large amounts of rock debris were generated by this unprecedented scaling effort and required special care in hauling and disposal (while under traffic flow conditions). Portable barriers and controlled stop and go traffic measures were employed to provide the utmost safety to the motorists.

Once the emergency contract was completed, a second and permanent contract was prepared for letting that following spring (1987). The contract called for the elimination of most all vertical cut slopes by slope flattening (Figures 19 & 20). In addition, the design called for the relocation of the two west bound lanes in the area of the large precipitous rock cut slope (where the recent fatality had occurred).

The permanent remedial contract called for the construction to be completed in two stages - repair of the east bound lanes by December, 1987, and repair of the west bound lanes by December, 1988. This was necessitated in order to maintain the heavy volume of east-west traffic. The December cut off date provided for non detoured traffic to flow unimpeded during the winter months.

Construction of the flattened cut slopes was accomplished by conventional excavation procedures using "dozers", "pans", and blasting where necessary. Most cut slopes were flattened from a vertical to a 1.5:1 to a 2:1 (Figures 21, 22, 23).

A plan to relocate the west bound lanes away from the base of the large 300′ high (91 meters) cut slope was enacted as part of this project. The roadway grade was raised 20 feet (6 meters) and the alignment shifted approximately 60 feet (18 meters) away from the base of the cut. This provided a depressed catchment ditch sufficient to handle most any anticipated rock fall. The relocated roadway and catchment ditch combined with

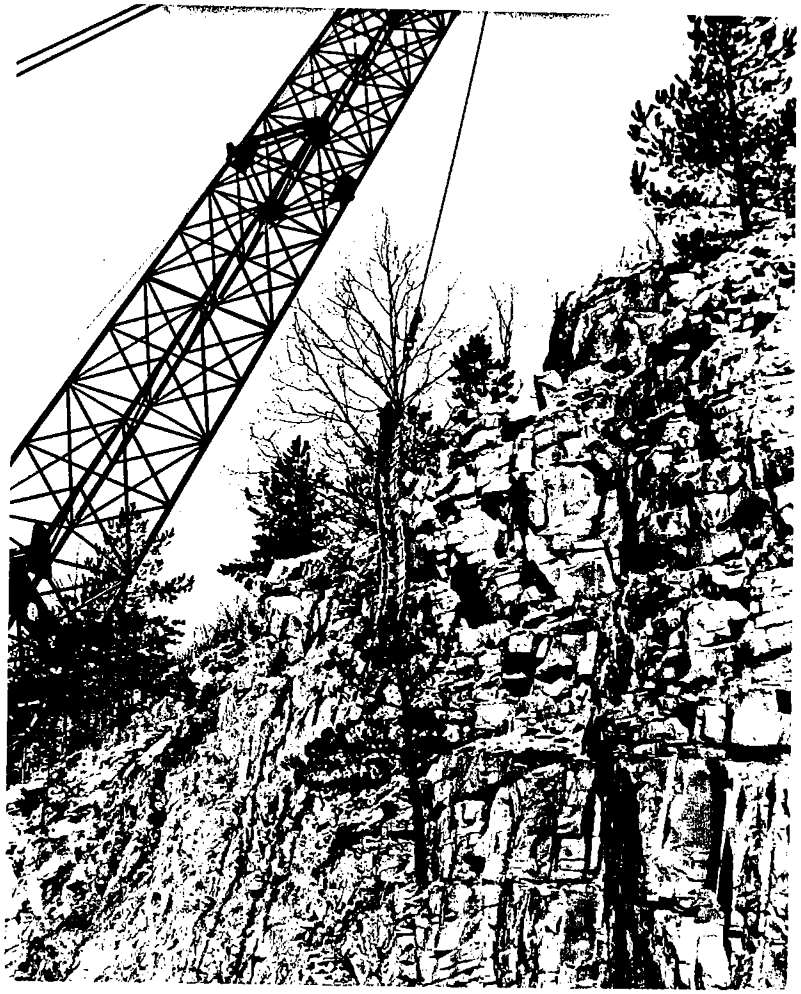


Figure 16. Cranes were used to aid in the scaling of rock slopes along I-40 in Cumberland County.

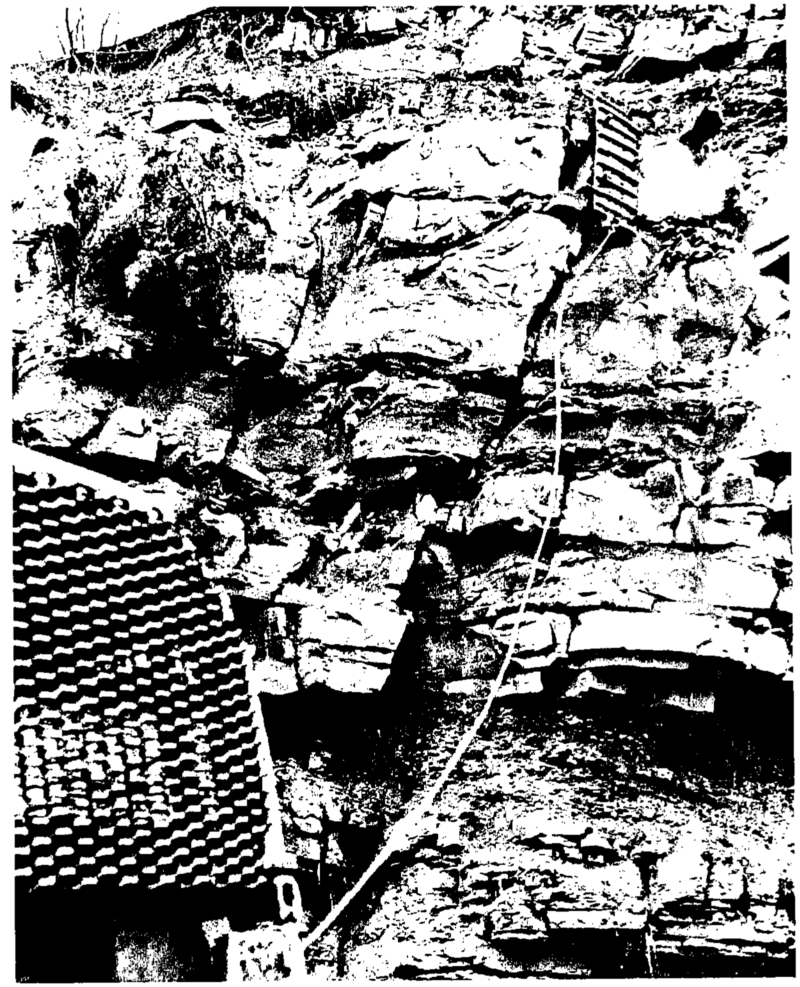


Figure 17. Dozer tracks were pulled back and forth across rock slopes as a method or scaling.

Figure 18. Portable catchment fences were used during the scaling project.

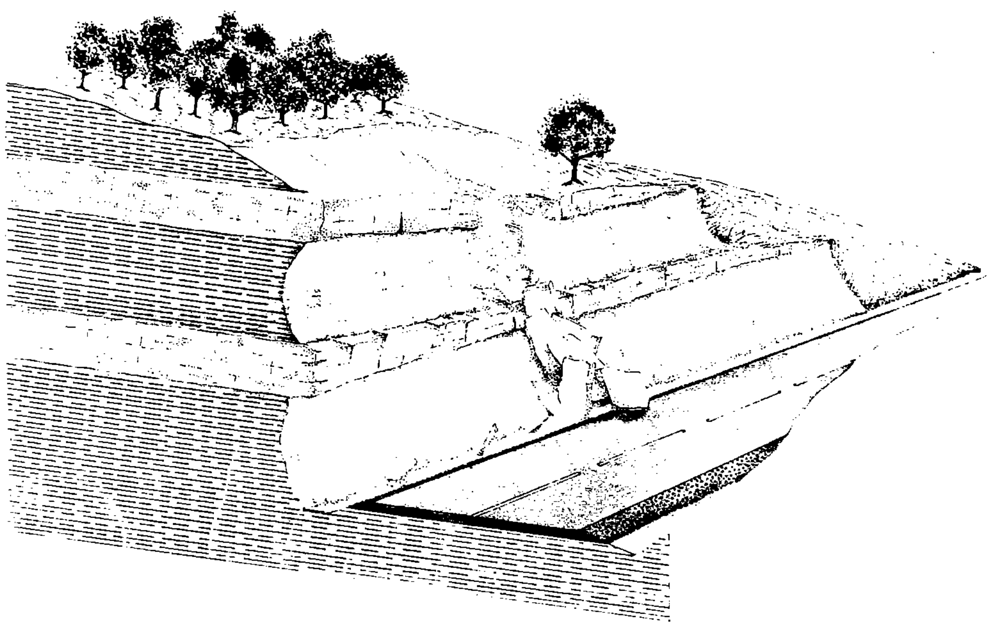


Figure 19. This diagram shows the typical rock slope condition along I-40 in Cumberland County prior to the remedial project.

the previous extensive scaling has proven to be a most successful concept (Figure 24, 25, 26).

A total of 1,311,767 cubic yards (1,002,977 cubic meters) of material was excavated from the cut intervals and subsequently placed in designated waste areas. The excavation price per cubic yard of material was $3.10. The total cost of the project was $5,704,824.19. Completion of the remedial work was accomplished in two years.

The overall project involved emergency scaling and trimming, permanent slope flattening, and partial realignment.

PROBLEMS/CONCERNS/BENEFITS

In evaluating geotechnical concepts it is necessary to properly identify the problems, concerns, and benefits. Although most of the remedial concepts discussed in this narrative are not new, they are however widely accepted and effective and deserve proper recognition.

Probably the item of most concern about the concepts discussed in this study is the safety issue of scaling under traffic conditions. To resolve the dilemma of scaling rock slopes alongside moving traffic, the use of portable protection barriers and fences were employed. A wide variety of barriers and fence types were used including "New Jersey" barriers, stacked metal shipping containers, and fences welded to I-beam runners which can be pulled from one location to another. Although these devices are not panaceas, they are, however, effective and most reasonably safe.

Also another technique used effectively on both projects was to stop traffic for 10 to 15 minutes at a time while the more precipitous rock slopes can be worked. It is imperative that effective traffic control devices such as lights, barrels, signs, and local police be used.

The obvious benefit of scaling precipitous rock slopes is the time needed to bring the cut slope to a safe condition (as compared to extensive redesign and excavation of cut slopes). Individual rock slopes can be adequately scaled in a few days. More extensive cut slopes can be scaled in a matter of a few weeks to a month or two. In emergency conditions rock slopes can be effectively "safed up" by scaling, providing the needed time for proper evaluation and possible redesign. Also, scaling an existing cut slope requires

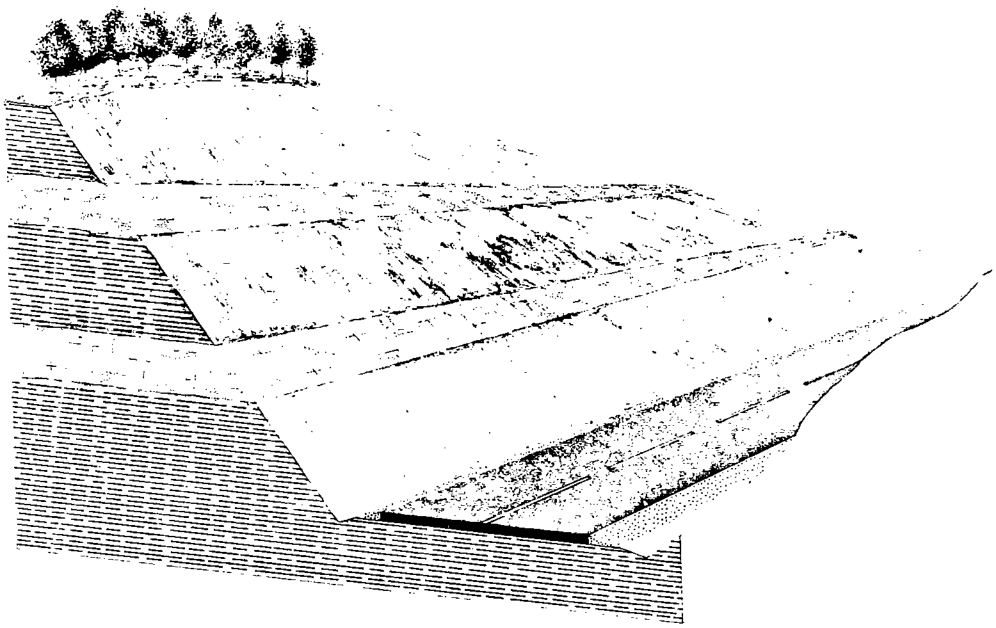


Figure 20. This block diagram illustrates the slope flattening concept (to reduce rockfall) as applied along I-40 in Cumberland County.

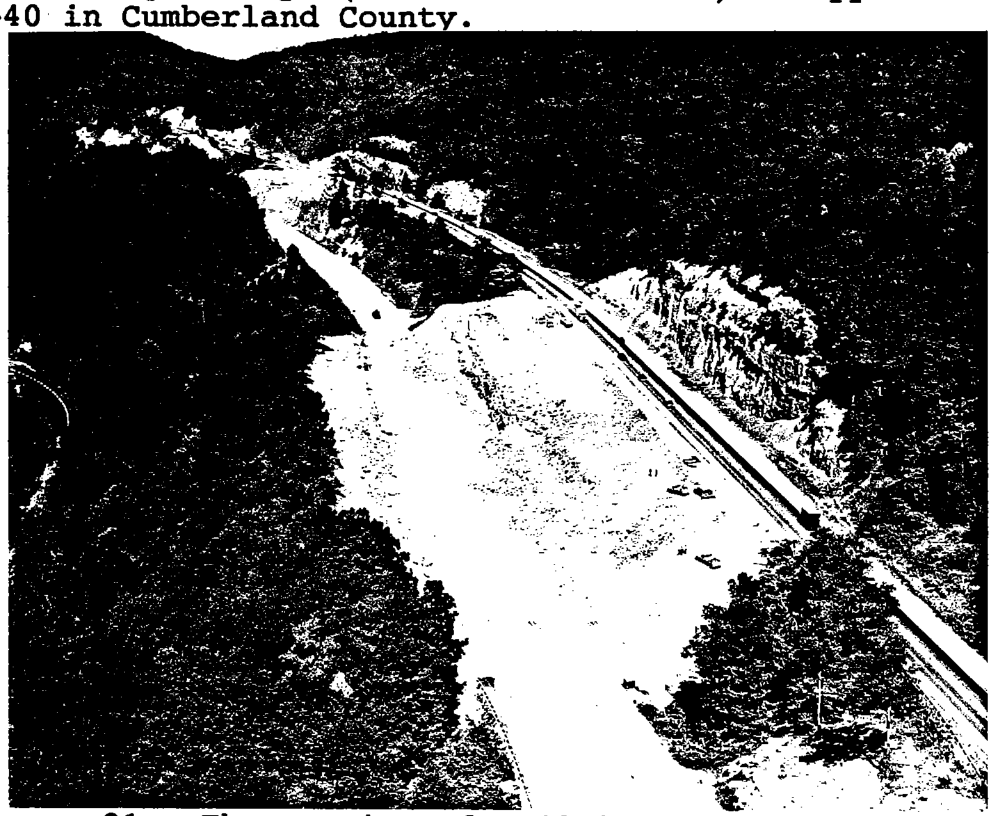


Figure 21. The section of I-40 in Cumberland County showing the precipitious rock cuts as the remedial project was initiated (photo by George Hornal).

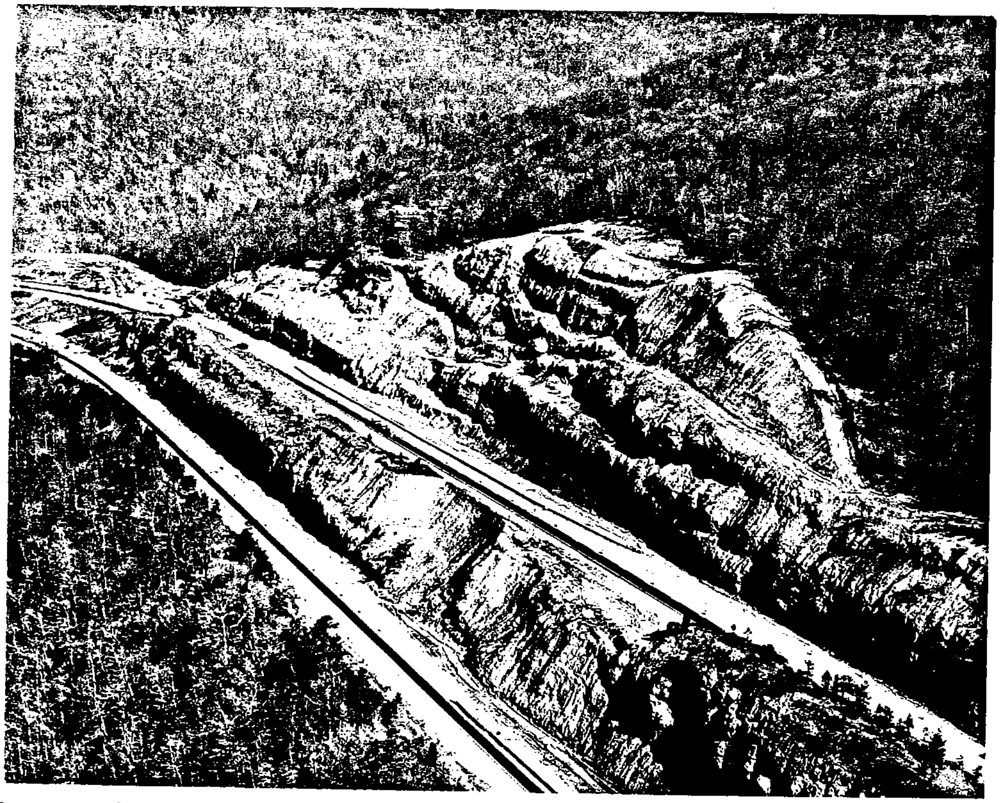


Figure 24. This photo shows the large 300′ high (91m.) cut prior to slope flattening of the median. Scaling of the high slope has been completed (photo by George Hornal).

Figure 25. The median cut section of the big cut interval was flattened and relocated to reduce rockfall events in the lower east bound lanes of I-40 (photo by George Hornal).

Figure 22. This photograph shows the result of the rockfall initiated remedial work; this is the "repaired" cut section shown in foreground (right) of Figure 21 (photo by George Hornal).

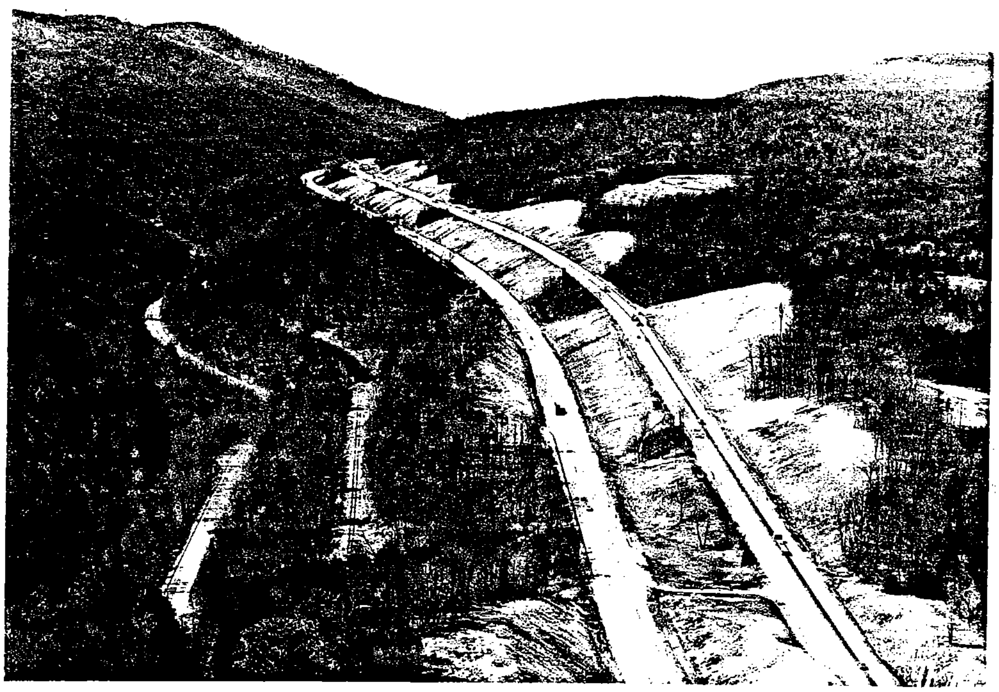


Figure 23. A completed section of I-40 in Cumberland County showing the results of the slope flattening concept (photo by George Hornal).

less time and involves less of an impact on the highway environment.

A drawback to scaling is that it does not provide a permanent cure to the problem. Continued weathering of the existing exposed rocks will eventually result in rockfall conditions becoming prevalent. The condition can reoccur within 5 to 7 years after cut slope rehabilitation. However, most slope conditions remain relatively safe for 10 to 15 years (depending on climatic conditions).

The obvious benefit of flattening the cut slope is that in most cases the rock fall problem is completely resolved. Again, this concept can be quite costly due to the excavation and disposal costs.

A major problem regarding the flattening of existing slopes is the disposal of the excess material; and in mountainous terrain usable sites can be difficult to obtain. In most cases waste areas have to be secured. However, the excess excavated material can be used to flatten existing fill slopes providing for the removal of guardrail sections.

CONCLUSIONS

The rehabilitation of precipitous cut slopes along certain sections of Interstate 40 in Tennessee have resulted in the development and successful implementation of innovative stabilization and protection techniques. The two projects documented in this paper have proven to be classic examples of successful rock slope remedial work.

The stabilization techniques tested and proven in this study include hand scaling and mechanical scaling. Some minor trimming was also used but not evaluated within this paper.

Another type of stabilization treatment which has proven to be exceptionally successful is the concept of flattening rock slopes. Although slope flattening has proven successful, the disposal of the waste material can be difficult if a waste site is not nearby.

Some of the protection devices successfully implemented include catchment fences, draped wire meshing, and rock fall control fences.

The two case studies discussed in this paper were chosen based on the differences between the two site locations, geologic conditions, and remedial concepts employed. The premise of this paper has been to share the results of unique engineering geology projects

Figure 26. The upper west bound lanes of I-40, Cumberland County were shifted 40 to 60 feet (12 to 18 m.) away from the high cut slope, providing an adequate catchment area (photo by George Hornal).

relating to rock fall problems along Interstate 40 in East Tennessee.

ACKNOWLEDGEMENTS

The writing of this paper required the help of valuable personel whom I, would like to acknowledge. Judy Sayne typed the manuscript for which I am gratefully thankful. The drawings which appear in this paper were drafted by Ken Brinkley and Jeff Snyder to whom I owe thanks. The aerial photographs were made by Gorge Hornal whose expertise is gratefully acknowledged. I also would like to acknowledge Mr. James Aycock, Tennessee Department of Transportation, for reviewing this manuscript and making helpful suggestions.

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Slope distress and rock fall induced by the presence of old underground excavations

Robert J. Watters
Geological Science Department, Mackay School of Mines, University of Nevada, Reno, NV 89557

Eric Rehwoldt
Geocon Inc., 6960 Flanders Drive, San Diego, CA 92121-2974

ABSTRACT

Rock fall and slope distress and instability occurred to excavated rock slopes, due to the adverse effects of old underground workings and fault zones being intersected by slope excavations. Mitigation and understanding of the problem was complicated as limited experience exists with designing slopes in rock masses with extensive underground workings. Detailed field investigations and laboratory testing enabled the extent of the underground excavations to be documented and the rock mass strength assessed. Two approaches were used to assess the problem. a) time lapse photography to show slope movements around the openings and b) two different types of slope models were utilized in the study, a two dimensional limit equilibrium model and a explicit finite difference model. Initial field observations suggested that total failure of the excavated slopes would occur, but review of the modelling results showed that this possibility would only occur when the underground excavations were analyzed with faults, though lower factors of safety do occur. The major effects of the underground excavations were to produce individual bench failures and localized slope distress which manifested itself as rock fall. These problems could be anticipated using the slope models provided the underground excavations were documented and the excavated slopes monitored.

INTRODUCTION

An interesting and possibly unique slope stability problem was encountered when old underground mine workings, excavated from the 1870's onward, Figure 1, were intersected by slope excavations. These mine workings adversely affected the slopes and caused signs of slope distress. These signs ranged from tension cracks, localized bench failures and unravelling slopes. Overall slope stability has to date not been affected. The main affect has

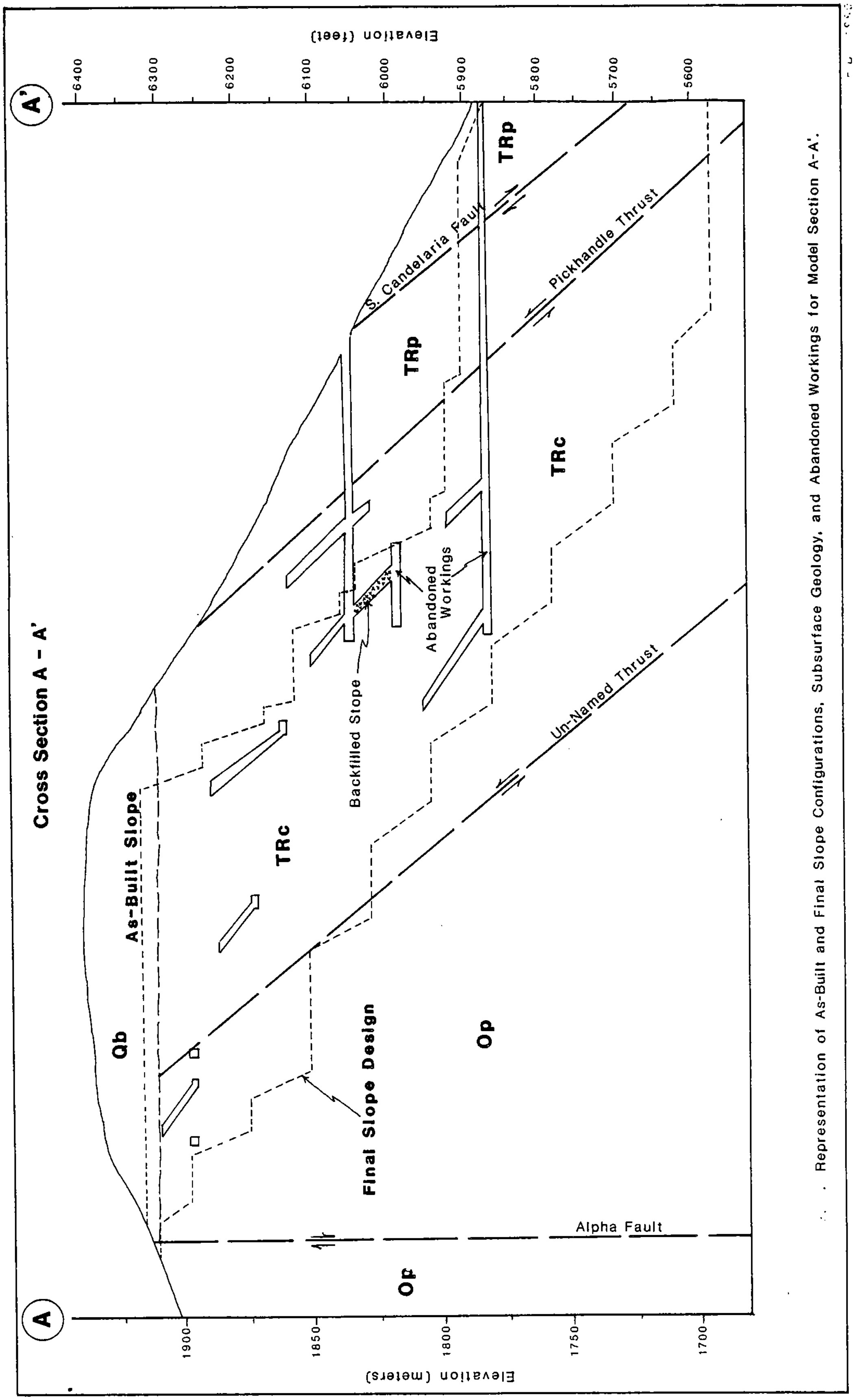

Figure 1. Cross-section through slope showing distribution of workings.

been to produce rock fall. A major contributor to the slope distress is the presence of extensive underground workings, which contain infilled stopes and large unfilled drifts and raises. The distribution of these large voids (up to 15 meters) and infilled stopes are only generally known. The distribution of such features influences bench stability, produces rock fall and a ragged slope. Little experience exists in dealing with this type of problem (Cotton and Matheson, 1989) as in most cases underground workings do not play a significant role in slope stability or rock fall. To understand the problem two approaches were utilized.

1. The first approached utilized time lapse photography over a period of weeks and months after slope excavation. The slopes were photographed from the same position and a photographic record was obtained of the changes in slope geometries, major areas of slope unravelling and rock fall.

2. The second approach utilized slope models based on a) two dimensional limit equilibrium techniques and b) explicit finite difference methods. The two methods were used to assess the physical condition of the slopes given the rock mass material property information and the presence of the underground openings. Comparison of the slope models permitted evaluation of the failure path sensitivity to the underground openings, and the deformation and the stress state of the excavated slopes.

Integrating both approaches gave insight into the slope deformation process and possible mitigation techniques.

SITE CONDITIONS

Geology. The topography at the mine site varies from gently rolling hills to rugged terrain due to active stream activity. Soil cover is variable with the majority of the hills covered with colluvium and the valleys infilled by alluvium. The rock lithologies consist of sandstones, shales, limestones, quartzites, and volcanic rocks, massive to thinly bedded, and cut locally by felsic dikes. Structurally, the bedding has a regional dip at 45 degrees to the north, but locally individual beds are highly contorted and dips and dip direction of the beds are highly variably. Major normal faults traverse the mine site striking approximately east-west with fault surfaces dipping to the south at between 50 to 60 degrees. All the faults contain variable thicknesses of fault gouge. The water table is below the present design pit bottom.

Rock mass strength and discontinuity characteristics. The rock mass behavior characteristics were developed from laboratory and field investigations. The laboratory study consisted of direct shear testing of rock joints and fault gouge, and triaxial testing and unconfined compressive strength testing of intact rock cores.The field study consisted of a site reconnaissance to assess the conditions of the as-built pit slopes and stable underground workings; testing for values of the in situ Young's modulus of deformation, E; sampling of representative intact rock, joints and fault gouge for laboratory testing; research/map checking of existing geologic and underground excavation maps.

The discontinuity cell mapping revealed three joint sets. One set dipping at between 35 to 40 degrees in a north-east direction, another set dipping at about 80 degrees approximately east south-east, and the strongest developed set striking approximately sub-parallel to the slope and dipping into the slope at 77 degrees.
The laboratory and in situ strength test results are summarized in Table 1.

Property	Rock Mass	Rock Fill	Fault Zone
E (Pa)	5.147e8	8.6e7	2.4e7
Poisson's Ratioo	0.25-0.30	0.35	0.40
Unit Wt.(kg/cu.m.)	2355	1842	2000
Cohesion (Pa)	2.39e6-1.40e7	----	2.7e4-5.5e4
Joint Friction (deg.)	32-36	36	13-30
Joint Cohesion (Pa)	1.24e5-3.06e5	----	-----
Rock Friction (deg.)	19-33	----	-----

Table 1. Rock mass material properties used in pit slope models.

TIME LAPSE PHOTOGRAPHY

Visual observation of the excavated slopes suggested that slope instability and rock fall came from regions surrounding the infilled and open stopes and drifts. Once these excavations were exposed by the slope excavations movements occurred as material caved into the open drifts and the infilled stopes unravelled. To confirm these observations and document the changes with time, photographs were taken shortly after individual benches were excavated where mine workings were exposed. These areas were re-photographed on a regular basis to permit comparisons to be made on slope changes with time. Comparing individual sets of photographs showed exactly where instability first started on the slope face and the method by which the instability progressed up slope.

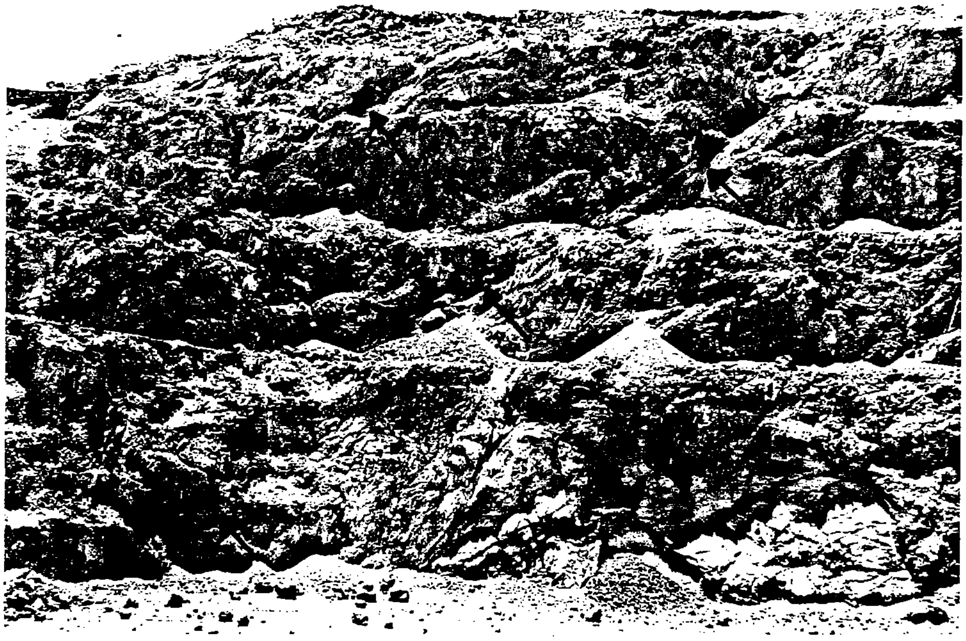

Figure 2. General photograph of complete slope.

Figure 2 shows a general photograph of the entire slope, approximately 600 feet in height. Two sets of underground workings are exposed trending diagonally from the top right side of the photograph to the bottom left. Rock fall can be seen in the toe region of the slope.

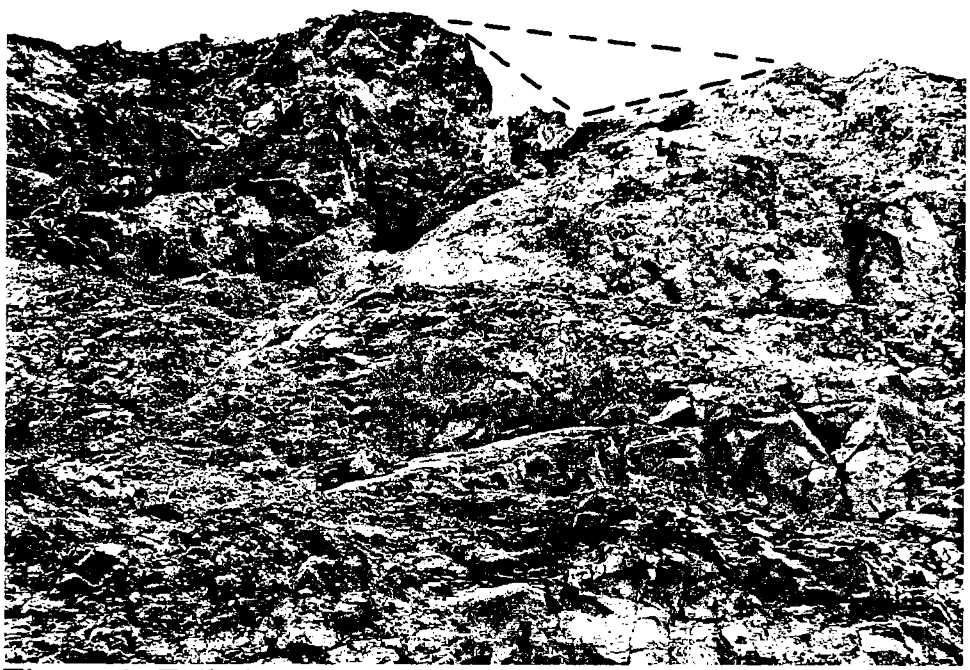

Figure 3. Failed unstable bench above underground excavations.

Figure 3 illustrates an unstable bench, part of which has already collapsed. The dashed outline shows the failed area immediately above exposed workings. To the left of the collapsed area a further rock mass can be seen to contain open fissures and is undergoing deformation. Typical unstable rock blocks are highlighted in Figure 4.

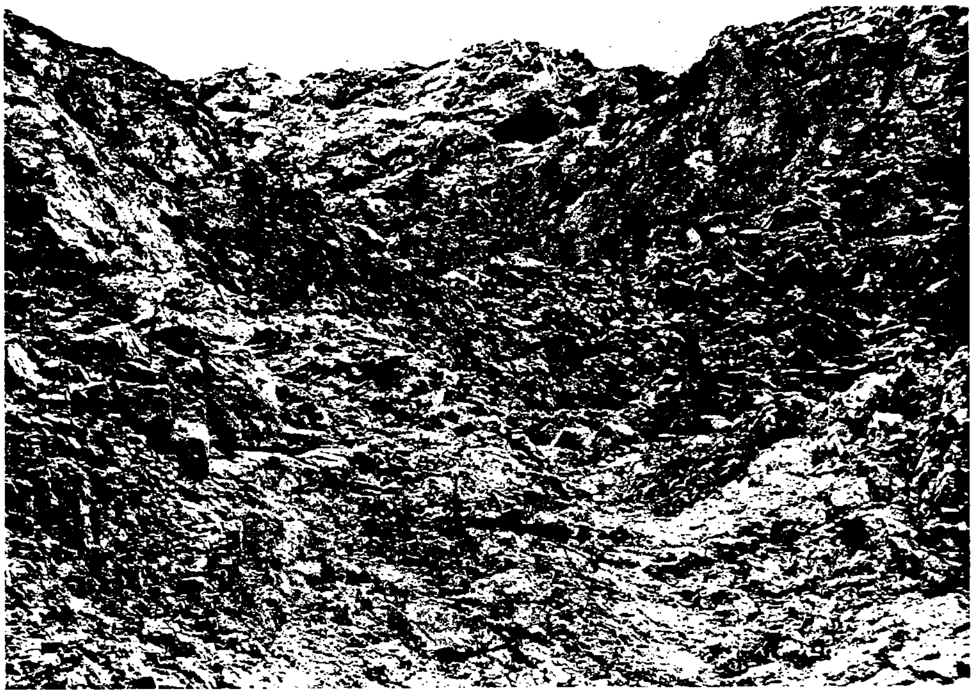

Figure 4. Unstable rock blocks, identified by arrows.

SLOPE MODELS

Two types of slope models were used to analyze the design slopes, a two dimensional limit equilibrium model and an explicit finite difference model. The results generated by the two models were compared to enable an understanding of the slope failure process to be developed. The slope model results evaluated the following concerns:

a) failure path sensitivity to the underground excavations and fault zones.
b) deformation and stress state of the excavated slopes.
c) safety factor for the overall slopes and benches.
d) rock mass strength sensitivity.
e) slope redesign.

Selected pit slope cross-sections for both as-built and end-of-pit configurations were based on the "worst case" distribution of underground workings and fault zone orientations. Modeling of sequential pit slope excavation was conducted in slopes most affected by the underground excavations. Pit slope behavior was examined by varying the material properties, degree of underground excavations, fault zone location and orientation, and slope height and angle.

Limit equilibrium analysis. STABLE 5, a two-dimensional limit equilibrium program was utilized for the factor of safety appraisal for the designed pit slopes. This program can deal with complex geometries, random searches and specified failure surfaces (Carpenter, 1986). The effects of the underground excavations on the stability of the overall slope was found to be minimal unless the underground excavations coincided with critically oriented fault zones and drifts. The most interesting aspect of the slope stability appraisal was in looking at individual bench stability and the effects of the underground excavations. In general the underground excavations were found to have a dramatic destabilizing effect on individual bench stability. A typical result can be seen in Figure 5 where in this case the factor of safety was found to be 0.76 for the slope geometry illustrated. Consequently the probability of failure is extremely high where underground excavations occur close to slope faces. This analytical solution concurs well with the observational signs of bench slope distress.

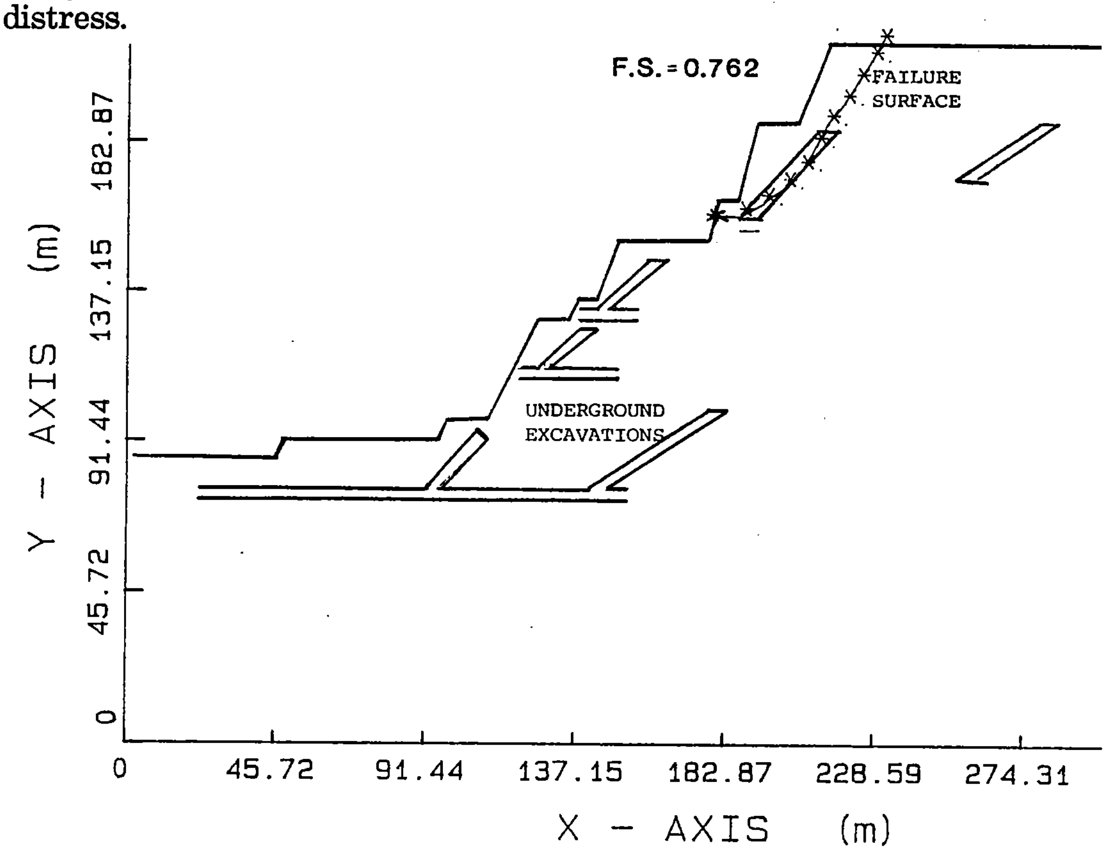

Figure 5. Bench instability failure surface, due to underground workings.

Finite difference analysis. A computer code, FLAC (Itasca, 1989), was used to assess the stress and deformation characteristics of the rock mass surrounding the underground workings and the overall slope. Initial stresses in the as-built slope model were generated by applying gravity loading to the topographic profile of the hillside prior to excavation. Lower and upper bound rock mass strength values were assigned to rock units and fault zones within the slope. Time-step calculations on the model were continued until a condition of static equilibrium was reached.

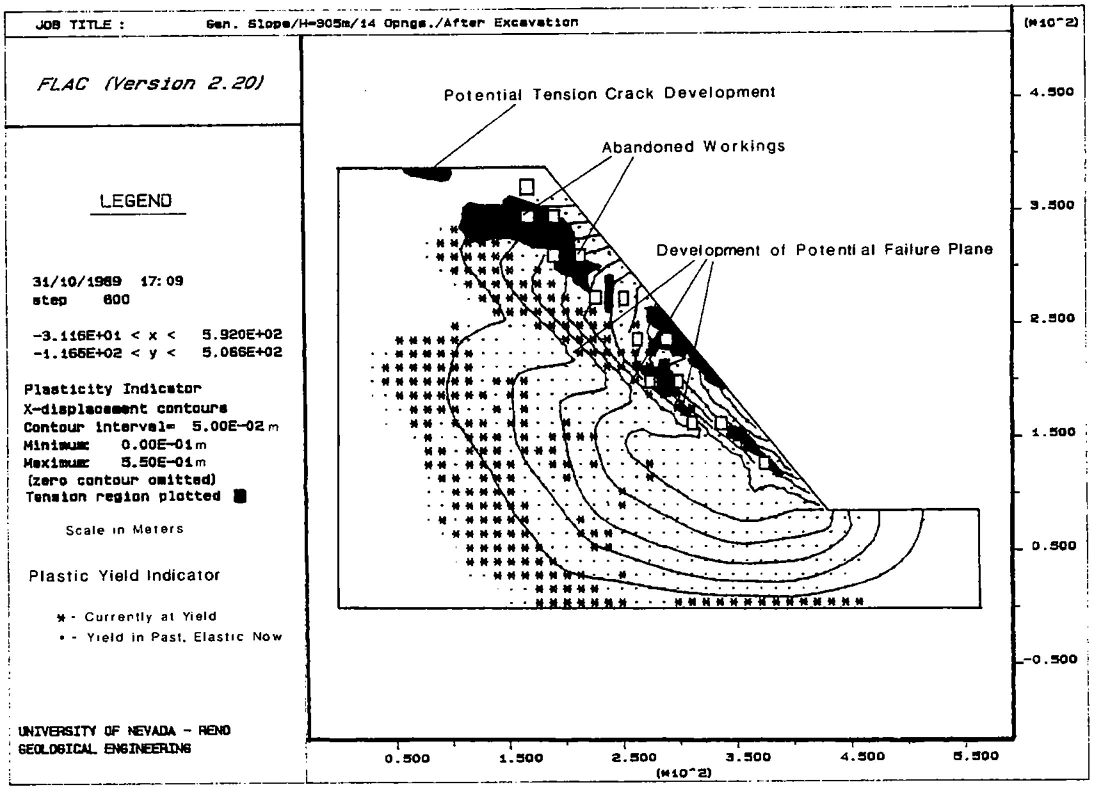

Figure 6. Finite difference model showing yielding joints and tension zones.

The finite difference modeling utilized different numbers of underground openings up to a maximum of 14 distributed near the slope face. Figure 6 shows the development of the potential failure surface, inferred by the anomalous pattern of lateral displacement contours. Tensional stresses can be seen to develop around the vicinity of the open stopes, and are confined to within the potential failing rock mass.

Joint and slope deformations are most prevalent within benches which are adjacent to either backfilled or open stopes, Figure 7. The largest tensile stresses are developed near the crest of individual benches and in the vicinity of abandoned workings. Lateral rock mass displacements in excess of 300mm was noted at the toe of some of the benches, suggesting possible failure at locations where the slope intersects drifts and stopes.

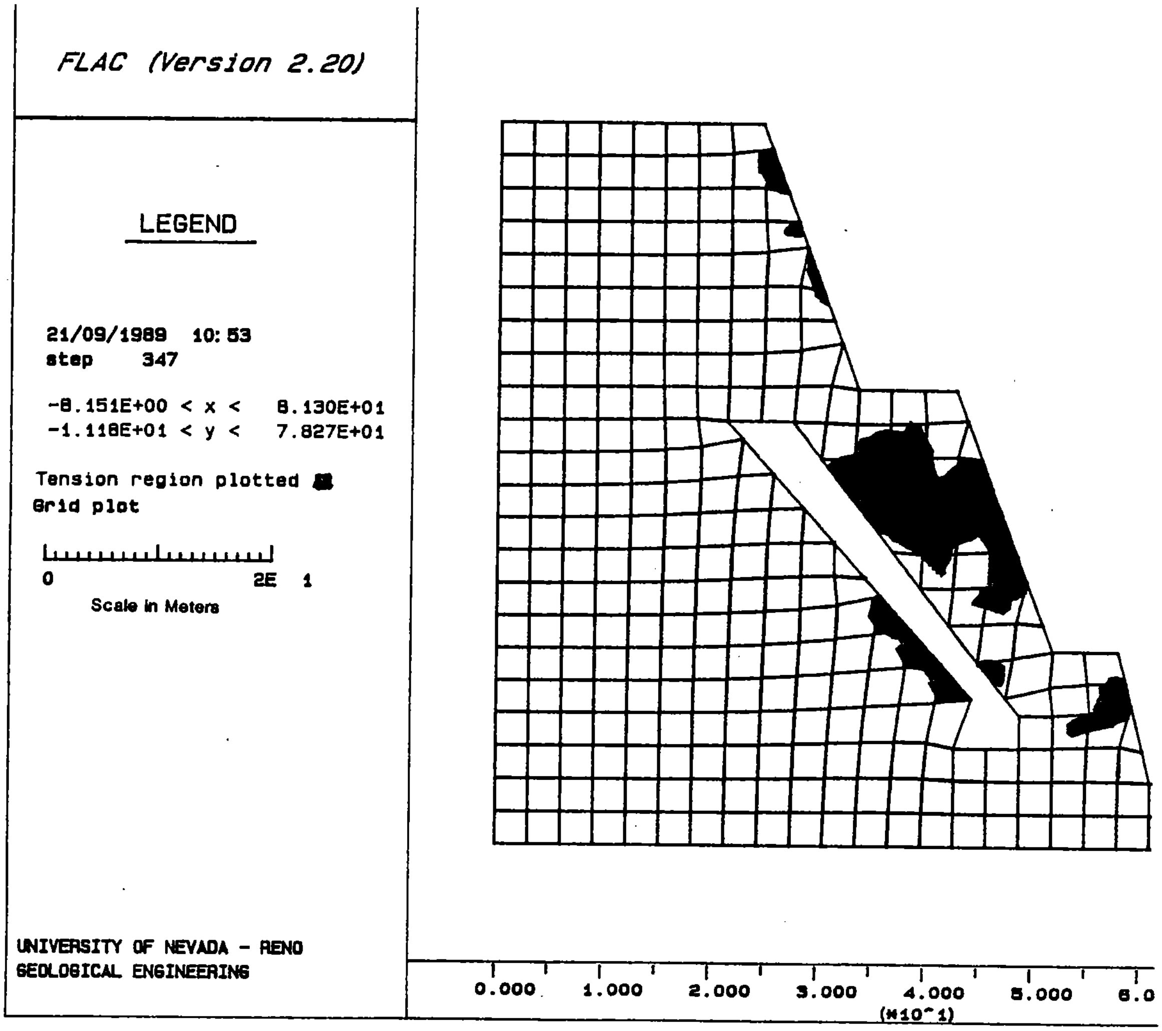

Figure 7. Tensional stress development induced by underground workings.

CONCLUSIONS

Limit equilibrium and finite difference methods indicate that the adverse effects of the underground workings will mainly be limited to bench instability. In general, overall slope stability is not affected unless a "worst case" scenario is used. This scenario would consist of a large number of critically oriented excavations together with faulting occurring in the same slope region. Tensile stresses and shear stresses were noted to develop around openings close to the slope face within the fractured rock. These stresses may cause bench failure, tension crack development and unravelling of the slope face. Slope monitoring is recommended to safeguard personnel and equipment.

Rock fall was found to be closely related to those regions of the slope where either underground excavations were exposed or backfilled stopes and raises were close to the surface. Openings of less than two meters were found to cause minimal problems. The presence of the mine workings affected slope stability by two processes.

1. The underground excavations when exposed by the slope excavations fail as the timber roof supports are destroyed. After the roof fails, migration of the void occurs up slope causing bench instability and associated rock fall.

2. The backfill easily erodes from the infilled stopes and raises when exposed by the slope excavations. This removes support from the surrounding rock which then fails and caves so permitting movement of the rock mass.

The majority of the slope problems, both rock fall and bench failures, were found to be more affected by erosion of backfill than destruction of roof timbers. This is thought to be a direct consequence of a progressive erosion with removal of support occurring over many months and destabilizing a large area of the bench. Whereas, roof timber destruction appeared to occur only close to the slope face, and once away from the slope face, stope and drift integrity were often found to be unaffected by slope excavations. Although this project was related to mine slopes, highway slope designers can be faced with similar problems.

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FIELD TESTS AND EVALUATION OF ROCKFALL RESTRAINING NETS

John D. Duffy, Engineering Geologist
Duane D. Smith, Engineering Geologist

California Department of Transportation
Office of Transportation Materials and Research
5900 Folsom Boulevard
Sacramento, California 95819

Abstract

The California Department of Transportation field tested and evaluated rock restraining nets engineered to absorb and dissipate rockfall impact energies to a maximum of 70 ft-tons. The nets were supplied by Brugg Cable Products (Switzerland) and Industrial Enterprise (France) and constructed by Caltrans personnel. Rocks were rolled down a 250-foot-long, 34-degree slope into the nets. Test rocks weighed from 300 to 13,000 pounds. Rock rolls were recorded on video and slow motion cameras for analysis.

Rockfall impact energy (total kinetic energy) was calculated by adding translational kinetic energy and rotational kinetic energy. Over 80 tests were conducted and analyzed. Rock net energy dissipation characteristics were analyzed. Based on field performance and energy analysis, modifications and adjustments in net design were made to reduce net maintenance. These nets stopped rocks delivering impact energies 1.5 to 2.5 times above the design load with acceptable levels of maintenance.

Maintenance and cleaning of the nets was easily accomplished by Caltrans maintenance personnel using normal maintenance equipment and supplies. Removal of rockfall debris can be accomplished by raising or lowering the net to allow access. Nets at road level can be cleaned with typical maintenance equipment. Damaged net components can usually be reused or repaired in a few minutes to two hours by a maintenance crew.

These wire rope rock restraining nets will become an integral part of the rockfall mitigation measures available for use along California's highways.

PHO 1-THREE TON ROCK #26 IMPACTING A BR G NET PANEL WITH 125 FOOT TONS OF EN GY

INTRODUCTION

The purpose of this research project was to construct, test, and evaluate the effectiveness of rock nets that will be used to mitigate rockfall hazards at selected sites in California. All aspects of the installation and performance, including field repair and cleaning, were evaluated and documented on video tape.

Apparently little, if any, actual field testing has been done where large boulders were rolled down natural slopes into rock nets. Therefore, little is known about the effects of rotational energy on individual components. Because available detailed data on the construction, performance, repair, and general maintenance of rock nets is not available, the decision was made to conduct our own tests and evaluations prior to installation of an actual project.

An excellent test site was selected on Coastal Highway 1 in Monterey County between Big Sur and San Simeon. It consisted of a 250-foot, 34-degree, unbroken slope free of heavy vegetation, with a relatively flat area at the base for installation of the rock nets.

Robert Thommen, Vice President and General Manager of the Santa Fe New Mexico office of Brugg Cable Products of Switzerland, provided a rock net for testing (less the structural posts) and the technical assistance necessary to construct their system. This system was constructed and tested by Caltrans personnel on August 8 through August 11, 1989 and November 12 through November 16, 1989.

Alain Lazard, Director of Diversified Ski Services of Squaw Valley, and the Untied States representative of Industrial Enterprise of Paris, France, provided their complete rock net for testing at this site. Their system was constructed and tested on December 4 through December 7, 1989.

Definition of a Rock Restraining Net

For purposes of this paper, a "rock restraining net" is defined as a rockfall protective device engineered to stop large rockfalls. The system consists of rectangular panels of woven wire rope vertically supported by steel posts and designed with frictional brake elements capable of absorbing and dissipating high energies. Both restraining systems utilize woven wire rope which has a fiber core providing greater flexibility than conventional steel core cable.

Description of a Brugg Rock Restraining System

The Brugg rock net constructed for the initial testing in August 1989 and November 1989 consisted of four woven wire rope panels supported by five steel posts (Figures 1 and 2). Individual panels were hung from a heavy perimeter wire rope supported by steel posts. The posts were set on concrete foundations secured by upslope and lateral anchor wire ropes. The upslope anchors and perimeter ropes were fitted with energy absorbing friction brakes.

This system was designed by Brugg to withstand 74 ft-tons of total kinetic energy with a safety factor of 1.5.

Description of the Industrial Enterprise Rock Net

The Industrial Enterprise (EI) rock net constructed for field testing in December 1989 consisted of three woven wire panels suspended from four box steel posts (Figures 3 and 4). Net panels were joined

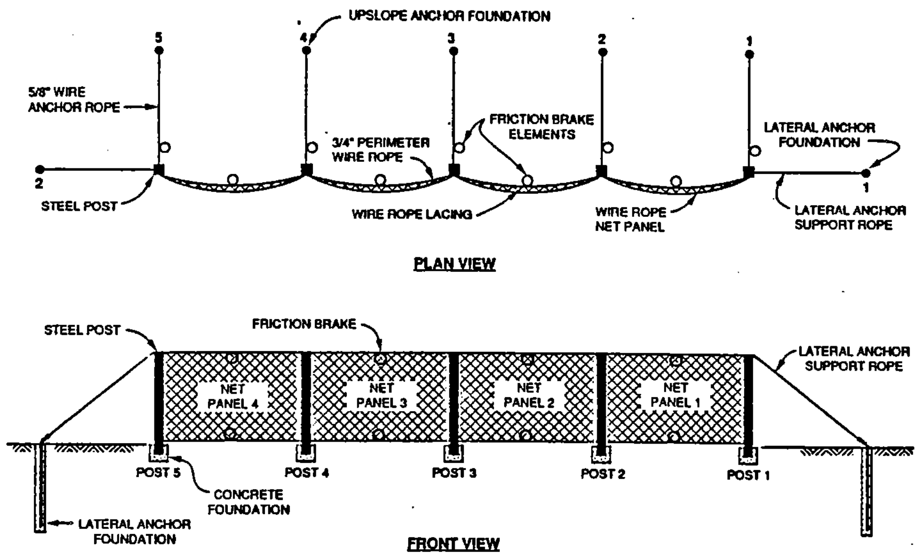

FIGURE 1. PLAN VIEW AND FRONT VIEW OF BRUGG ROCK NET CONSTRUCTED FOR THE INITIAL FIELD TESTING

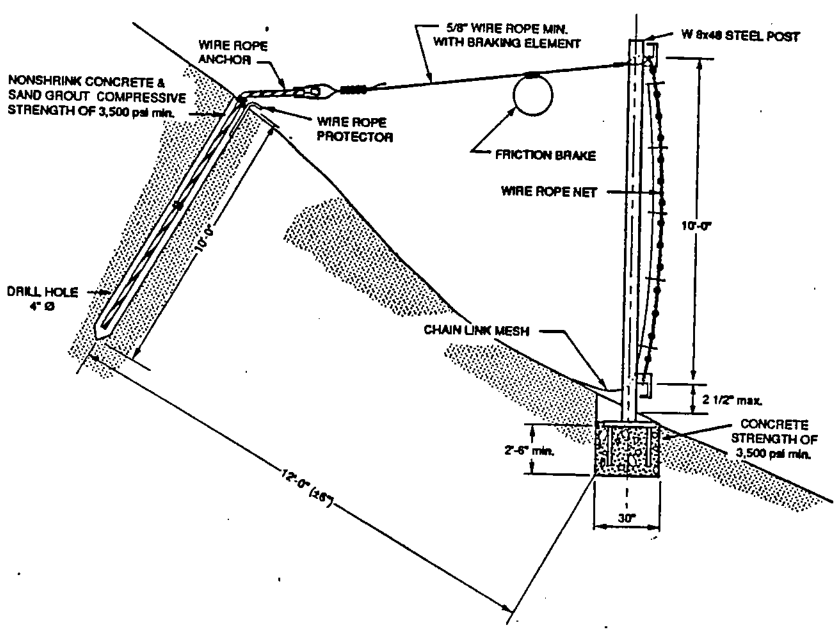

FIGURE 2: SIDE VIEW OF A BRUGG BASEPLATE POST FOUNDATION WITH UPSLOPE ANCHOR SUPPORT

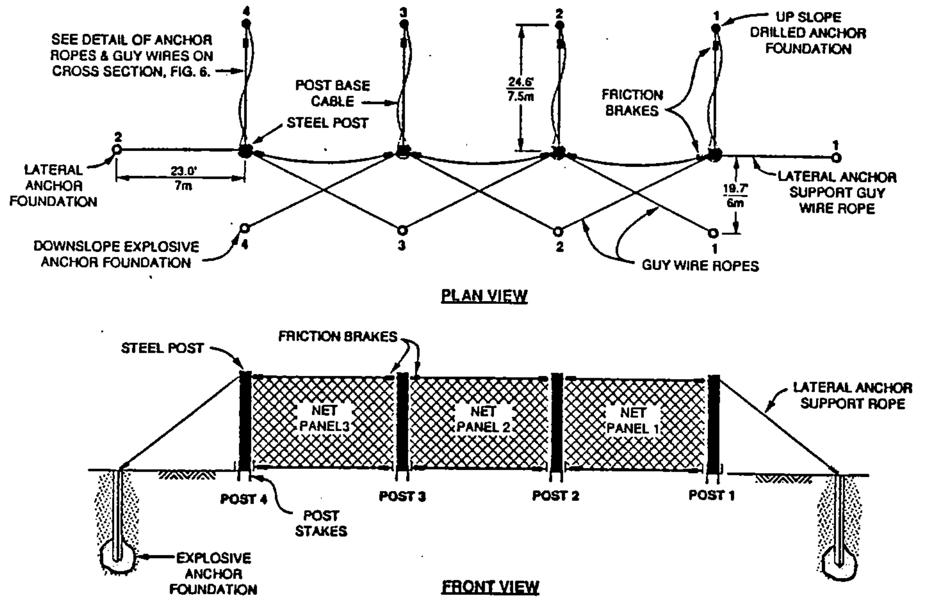

FIGURE 3. PLAN VIEW AND FRONT VIEW OF THE INDUSTRIAL ENTERPRISE ROCK NET CONSTRUCTED FOR THE INITIAL FIELD TESTING

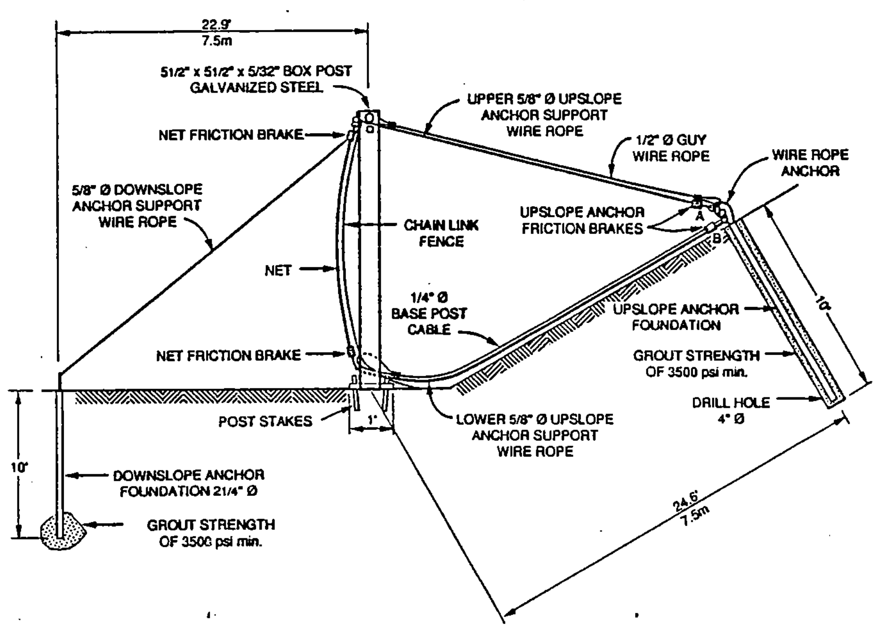

FIGURE 4: SIDE VIEW OF THE INDUSTRIAL ENTERPRISE ROCK NET

together on their common sides and linked together at their corners with chain. Wire rope attached to the chain passed through the top of the box posts to the friction brakes attached to the uphill anchors. Friction braking elements were also attached to each corner of the individual panels. Three guy wire ropes were attached to the top of each steel post for support. One was secured to an upslope anchor and the other two to downslope anchors. Guy wires also extend from the end posts to the lateral anchors located at each end of the rock restraining system.

A 70 ft-ton rock restraining system was requested from EI for testing.

TEST PROCEDURE

The test slope is 130 feet high and 100 feet wide with an overall slope angle of 34 degrees ($1\frac{1}{2}$:1). The slope measured along the ground surface is 250 feet long. A cross-section was developed from a detailed survey of the slope (Figure 5). Survey points were located two to five feet apart. Relative to the rockfall diameters, the slope is smooth and did not greatly affect rockfall trajectories. There are, however, several gullies which affected rockfall trajectories of small (one to two feet in diameter boulders). Vegetation is sparse and had little, if any, effect on rockfall trajectories.

The slope material is composed of landslide debris consisting of 1- to 18-inch rock fragments in a matrix of clayey silt. This material was dry and hard during all three tests. However, in some areas, successive boulder rolls broke up the surface creating soft spots which slowed some boulders. It was observed during the tests that, in most cases, rocks revolved around the shortest axis for the first 150 feet. As rock velocity increased, rocks then revolved around their longest axis.

Test boulders were obtained from a local stockpile of rockfall and rockslide material. These boulders are dense greenstone with a specific gravity ranging from 2.91 to 3.03. Eighty boulders were rolled and identified by a name or number.

Prior to rock rolling, the three principle axes (x, y, and z) of each boulder were measured. These values were used to estimate rock weight and inertia. Fifteen boulders were accurately weighed with a load cell. Actual weights were compared to estimated weights to evaluate estimated weight accuracy.

Rock rolling was recorded on video and high-speed (16 mm) film from four different locations along the slope. Real time was recorded on video (30 frames per second) and slow motion coverage was recorded on high-speed film (60 to 80 frames per second). The four cameras captured two wide views, one oblique, and one frontal.

In order to fully utilize the film and video footage, reference lines on 50-foot intervals were placed on the slope perpendicular to the slope axis. During the Industrial Enterprise test (Test 3), the spacing of reference lines on the lower portion of the slope was modified. This allowed detailed measurements of boulder travel time over a known distance. The information was used to calculate rockfall velocities. Yellow, three-inch-wide "caution" tape was used for the reference lines because of its high visibility. In addition, stadia rods, three feet to six feet high, were randomly placed on the slope for bounce height analysis.

Rocks were dropped from a height of 10 feet over the edge of the slope with a front end loader. Extremely large boulders (Test 2, Rolls No. 37 and 38; Test 3, Rolls No. 10 and 12) and the simulated rockslide (Test 3, Roll No. 19) were pushed off the edge of the slope with a front end loader.

All data were recorded for each rock roll on a data sheet developed by Transportation Materials and Research (TM&R) staff (Figure 6). Recorded data included impact locations, net damage, and net

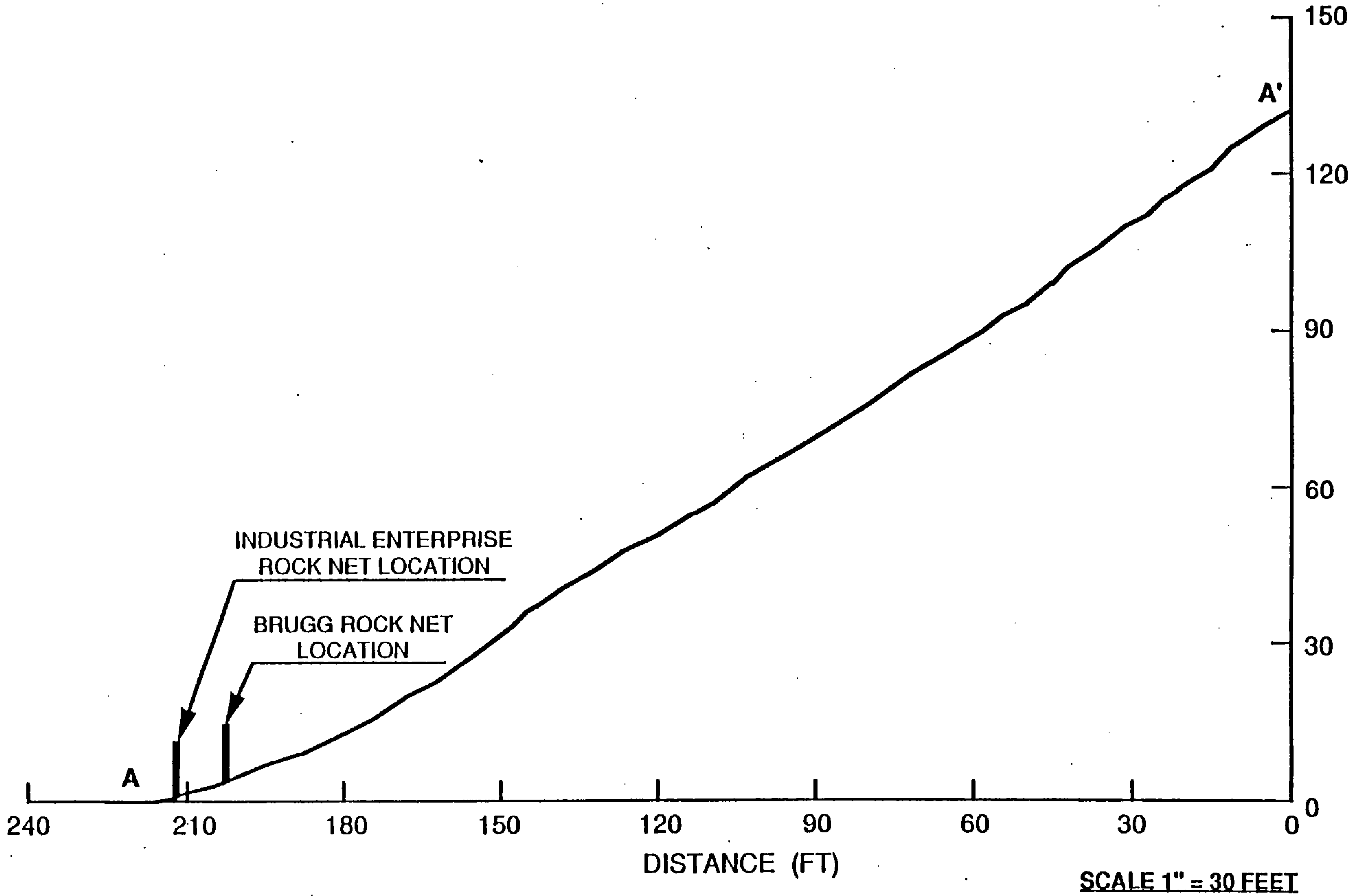

FIGURE 5: CROSS SECTION OF THE TEST SLOPE

ROCK RESTRAINING NET TEST DATA SHEET

MANUFACTURER: ________ VIDEO I.D.: ______ PAGE: /

TEST OR ROLL#: ________ TEST #: ______

ROCK I.D.: ________ ROLL #: ______

A. ROCKFALL DATA

1. DIMENSIONS: ________ METERS(m) ________ FEET(ft)
2. VOLUME (V): ______ CUBIC METERS ______ CUBIC FEET
3. UNIT WEIGHT( ): ______ Kg/m^3 ______ Lb/ft^3
4. WEIGHT(W) ______ Kg ______ Lb
5. MODE OF TRAVEL BEFORE IMPACT: (ROLL,BOUNCE,SLIDE,FREEFALL) ________

WEIGHT
ESTIMATED
1. ________
2. ________
3. ________
REAL ________
% ERROR ________

B. MOMENT OF INERTIA (I):

1. SHAPE: ________
2. AXIS OF ROTATION: ____
3. I= ________

C. VELOCITY:

1. TRANSLATIONAL
   a. LENGTH OF ROLL IN TIME t ______ m ______ ft
   b. TIME t ________ sec
   c. VELOCITY v ______ m/sec ______ ft/sec
   d. LENGTH OF ROLL IN TIME t ______ m ______ ft
   e. TIME t ________ sec
   f. VELOCITY v ______ m/sec ______ ft/sec

2. ANGULAR
   a. ROTATION: ______ degrees
   ______ radians
   b. TIME: ______ secs
   c. ANGULAR VELOCITY( ): ______ sec^{-1}

D. ENERGY K.E. = 1/2 m v + 1/2 I

a. ACCELERATION OF GRAVITY(g) = ______ m/sec ______ ft/sec

b. MASS m= W / g ______ $Kg/m/sec^2$ ______ $Lb/ft/sec^2$

c. TRANSLATIONAL K.E.= 1/2 m v
   1. maximum ________ Kilojoules ________ Foot-tons
   2. minimum ________ Kilojoules ________ Foot-tons

d. ANGULAR K.E.= 1/2 I
   1. maximum ________ Kilojoules ________ Foot-tons
   2. minimum ________ Kilojoules ________ Foot-tons

e. TOTAL KE = TRANSLATIONAL + ANGULAR.
   1. maximum ________ Kilojoules ________ Foot-tons
   2. minimum ________ Kilojoules ________ Foot-tons

E. REMARKS

ROCK NET DATA:

1. ROCK STOPPED BY NET ______ YES ______ NO
2. HEIGHT OF IMPACT IN THE HET PLANE ______ m ______ f t
3. REMARKS:

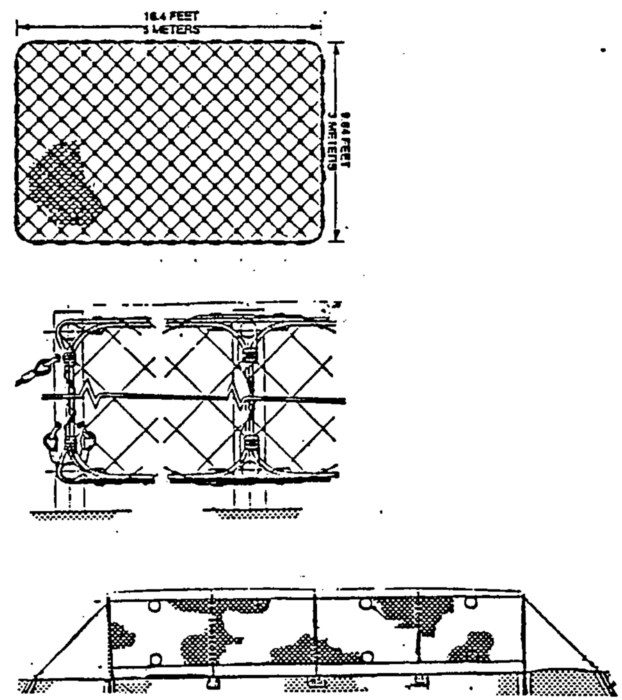

FIGURE 6: SIDE VIEW OF THE INDUSTRIAL ENTERPRISE ROCK NET

repairs. With over 80 rock rolls, this system was essential for recording and organizing the large amount of data obtained.

Rockfall Energy Analysis

Kinetic Energy

Kinetic energy is the most common measurement used to describe rockfall for engineering design. Throughout the energy analysis, each rockfall was treated as a rigid body in motion. According to Chasles' theorem, any general displacement of a rigid body can be represented by a translation plus a rotation (2). Based on this theorem, the process of rockfall is made up of two components: translational motion and rotational motion. These two components can both be quantified as energy in motion, or kinetic energy. Calculation of these kinetic energies (KE) is based on the assumption that the mass of the boulder is concentrated at the center of mass and its motion revolves around the center of mass (2). Rockfall motion is therefore the sum of the translational kinetic energy (KE_T) and the angular kinetic energy (KE_A) (1, 2, 6, and 7). This sum, the total kinetic energy (KE), is expressed mathematically as:

$$\text{Total KE} = KE_T + KE_A = 1/2\, mv^2 + 1/2\, I\omega^2 \qquad \text{Eq. 1.1}$$

where m is the mass of the boulder, v is the velocity of the boulder just before impact, I is the moment of inertia of the boulder as it spins, and ω is the angular velocity of the spinning boulder just before impact.

Calculations

Each rockfall impact is measured in terms of foot-tons of total kinetic energy. Maximum value calculations use 110% of the estimated weight and minimum value calculations use 90% of the estimated weight. A computer spread sheet was developed by TM&R staff to calculate the total kinetic energy. The results of these calculations are in Tables 1, 2, and 3.

Dynamic Load Path Analysis

A dynamic load path analysis was performed in an attempt to determine the forces occurring within individual net components. This information is used to balance the net system so that each component will function without failure.

Rocks impacting the net generate forces throughout the net system which are dissipated through the flexibility of the net. These forces, emanate from the point of impact to the net system perimeter and apply loads that travel along a "load path" (Figures 7 and 8). The load path consists of several structural net components with different strengths and load-dissipation capabilities (Figure 9). When all of the components in the load path are in equilibrium, the net system is "balanced". A balanced net system is the optimum design for load-carrying capacity.

Three rockfall impacts were analyzed dynamically to identify the load path and the loads within the load path. This was accomplished by analyzing the film footage of actual tests and using those data in the calculations.

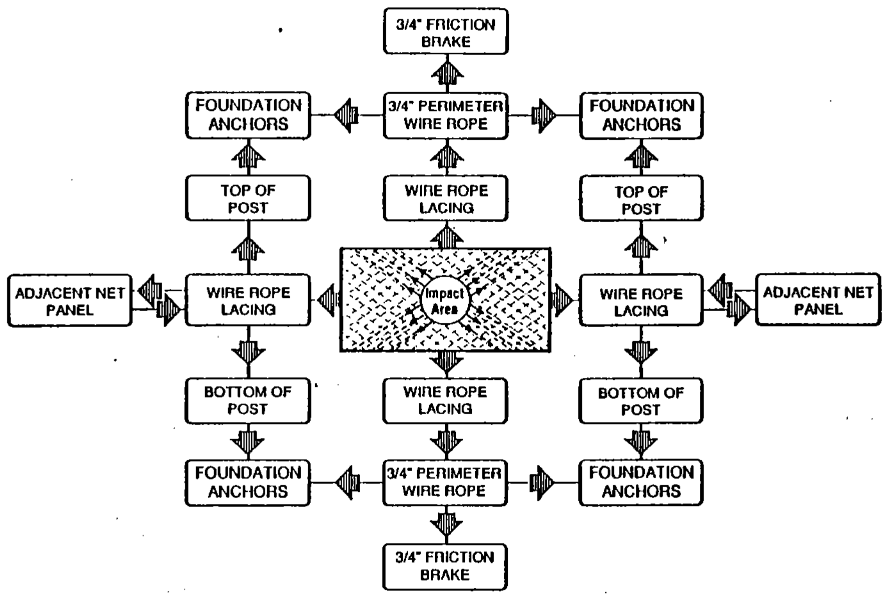

FIGURE 7: FLOW CHART OF THE BRUGG NET SYSTEM LOAD PATH.

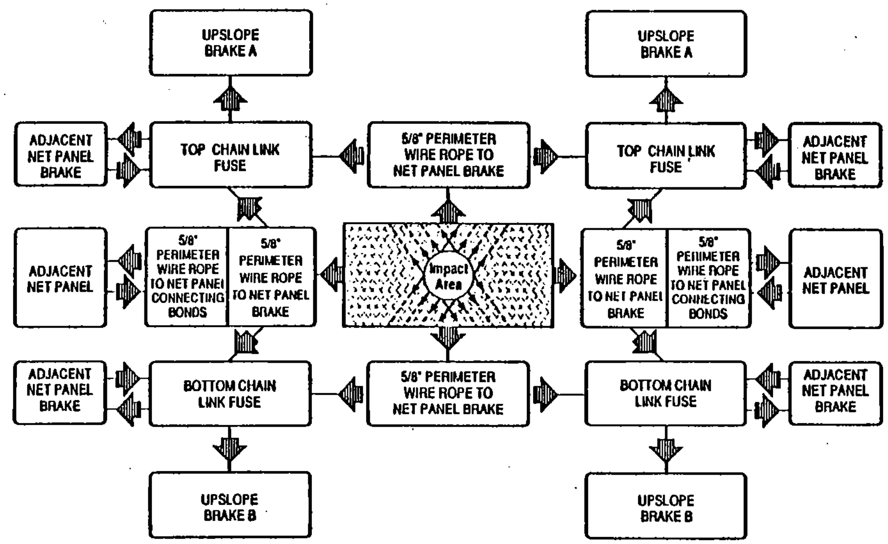

FIGURE 8: FLOW CHART OF THE INDUSTRIAL ENTERPRISE NET SYSTEM FLOW PATH

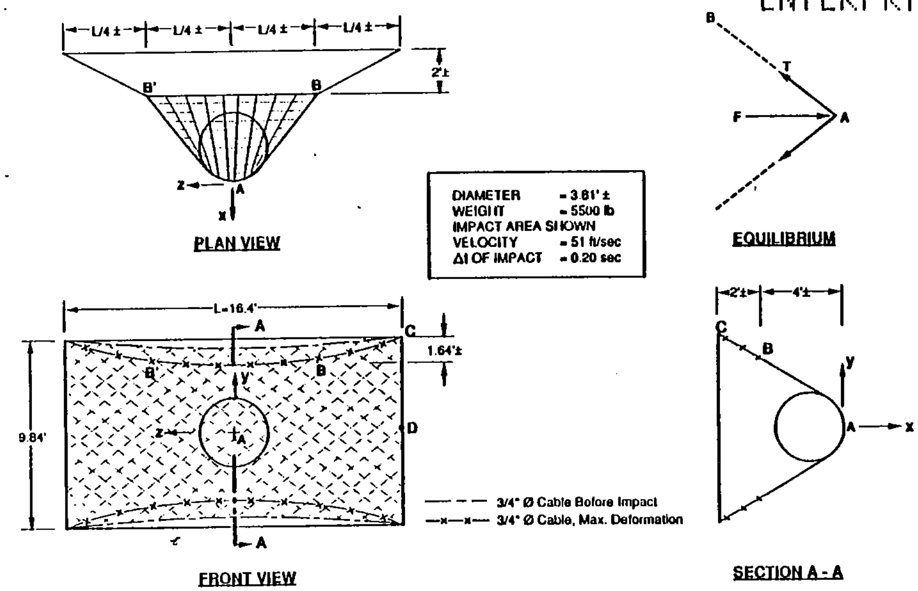

FIGURE 9. GEOMETRY OF DEFORMATION AND EQUILIBRIUM AT THE POINT OF CONTACT OF THE ROCK BODY.

Table 1: Weight, Velocity, and Energy Table for Brugg Test No. 1

Rock Roll Number	Weight (lb) max. min.	Total KE (ft-tons) max. min.	Rock Stopped	Remarks
1	MISSED			
2	319 261	4.6 3.7	Yes	
3	511 419	5.5 4.5	Yes	
4	589 482	9.8 8.1	Yes	
5	880 720	19.3 14.4	Yes	
6	921 754	6.3 5.2	Yes	Stopped upon initial impact but upon rebounding, hit stationary rock, pushing it through the net.
7	MISSED			
8	1172 959	46.2 37.8	Yes	
9	MISSED			
10	2021 1653	37.9 31.0	Yes	Stopped upon initial impact but upon rebounding, hit stationary rock, pushing it through the net.
11	MISSED			
12	1749 1431	6.1 5.0	Yes	
13	1504 1232	7.3 6.0		
14	MISSED			
15	*1700	35.4	Yes	
16	*1860	13.8	Yes	
17	MISSED			
18	2354 1926	23.0 18.8	Yes	
19	7091 5802	164.4 134.5	Yes	

*Actual weight.

All other weights based on estimated volume times unit weight ± 10 percent for maximum and minimum values.

Table 2: Weight, Velocity, and Energy Table for Brugg Test No. 2

Rock Roll Number	Weight (lb) max. min.	Total KE (ft-tons) max. min.	Rock Stopped	Remarks
1	MISSED			
2	301 246	1.9 1.6	Yes	
3a	140 114	1.5 1.2	Yes	
3b	209 171	2.2 1.8	Yes	
4	535 437	11.5 9.4	Yes	
5	883 722	21.3 17.4	Yes	
6	342 280	4.0 3.2	Yes	
7	376 307	4.6 3.8	Yes	
8	MISSED			
9	1254 1026	3.4 3.4	Yes	
10	MISSED			
11	*950	8.2	Yes	
**12a	*606	2.9	Yes	
12b	*1820	14.9	Yes	
13	*1590	24.7	Yes	
14	*1590	21.7	Yes	
15	3788 3099	42.5 34.8	Yes	
16	2299 1881	26.6 21.8	Yes	
17	MISSED			
18	*1700	17.8	Yes	
19	505 413	7.3 5.9	Yes	
20	619 506	1.3 1.0	Yes	
21	MISSED			

Table 2 (Continued): Weight, Velocity, and Energy Table for Brugg Test No. 2

Rock Roll Number	Weight (lb) max. min.	Total KE (ft-tons) max. min.	Rock Stopped	Remarks
22	292 239	.8 .8	Yes	
23	607 496	4.5 3.7	Yes	
24	717 586	1.2 1.0	Yes	
25	MISSED			
26	MISSED			
27	MISSED			
28	MISSED			
29	MISSED			
30	917 750	16.8 13.7	Yes	
**31a	773 633	17.4 14.3	Yes	
31b	773 633	10.9 8.9	Yes	
32	*2630	70.0	Yes	
33	MISSED			
34	1052 862	8.4 6.9	Yes	
35	3148 2576	70.3 57.5	Yes	
36	*5500	125.4	Yes	
37	*12700	295.3	No	Rock energy was attenuated causing the rock to stop 41 feet downslope.
38	10972 8977	232.9 190.5	Yes	

*Actual weight.

**Rock Rolls No. 12 and 31 split and each portion is recorded as "a" and "b".

All other weights based on estimated volume times unit weight ±10 percent for maximum and minimum values.

Table 3: Weight, Velocity, and Energy Table for Industrial Enterprise Test No. 2

Rock Roll Number	Weight (lb) max. min.	Total KE (ft-tons) max. min.	Rock Stopped	Remarks
1	*1590	27.6	Yes	
2	*1860	17.1	Yes	
3	1746 1430	16.4 13.5	Yes	
4	1793 1467	8.2 6.7	Yes	
5	MISSED			
6	1571 1287	17.0 13.9	Yes	
**7a	916 750	5.7 4.6	Yes	
7b	916 750	11.5 9.4	Yes	
8	*2630	34.9	Yes	
9	*5500	67.1	No	Rock hit Post 1 and rolled to the right where there was no net.
10	11182 9149	116.2 95.1	Yes	
11	*2630	56.3	No	Rock hit Post 4 and rolled to the left where there was no net.
12	13845	184.3	No	The high energy caused the net to lay down allowing the rock to pass over the net.
13	MISSED			
14	MISSED			
15	7487	86.6	No	The net was sagging three feet above ground and the rock rolled over.
16	MISSED			
17	MISSED			

Table 3 (Continued): Weight, Velocity, and Energy Table for Industrial Enterprise Test No. 2

Rock Roll Number	Weight (lb) max. min.	Total KE (ft-tons) max. min.	Rock Stopped	Remarks
18	1989 1627	23.8 19.5	No	The net was sagging three feet above ground and the rock rolled over.
***19a	2257 1847	12.7 10.4	Yes	
19b	1692 1386	7.2 5.9	Yes	
19c	762 624	1.6 1.3	Yes	
19d	167 137	0.7 0.6	Yes	

*Actual weight.

**Rock Roll No. 7 split in half and each half is recorded as 7a and 7b.

***Rock Roll No. 19 was a simulated rockslide where rocks a, b, c, and d were rolled simultaneously.

All other weights based on estimated volume times unit weight ±10 percent for maximum and minimum values.

Such events are analyzed using the vector quantities of momentum, and impulse:

$$F\,t = m\,v \qquad \text{Eq. 1.2}$$

Impulse = momentum change

where F is the applied force, t is the time it takes the boulder to stop, m is the mass of the boulder, and v is the translational velocity at the initial point of impact.

This analysis shows that under these conditions, the load per 100% loaded net strand is 4463 lb. The load in the top and bottom perimeter wire rope, where the load dissipating friction brakes are located, is 26421 lb in the outer thirds of the wire rope and 11425 lb in the middle third of the wire rope and the load in the upslope anchor cable is 11828 lb.

This approach can be used as a guide to designers in selecting the net grid spacing and minimum tensile strength of the friction brakes.

Friction Brake Analysis

Several laboratory tests were conducted on the friction brakes to evaluate their performance. Tests were performed on new brakes, retorqued brakes, and two brakes in tandem. An MTS electro hydraulic machine with a one million pound capacity was used to load the brakes until they activated.

Brugg tensile brake strengths were within ± 8% of the factory specifications. EI tensile brake strengths were 4% to 30% above factory specifications. This variation between specified and laboratory-tested brake tensile strength is acceptable. Variability is expected to occur because of the cable flexibility. Over time, especially when transported, the cable will reseat itself in the brake sleeves, changing bolt torque and, to some degree, brake tensile strength. Bolt torque was less after movement of the cable through the brake in every test.

MAINTENANCE OF ROCK NET SYSTEMS

Considerable interest has been expressed by maintenance personnel throughout the state concerning the level of repair and methods of cleaning required for rock nets. Although it is generally accepted in Caltrans that this mitigation measure effectively stops rockfall, maintenance requirements are considered important in the practical use of the nets. Therefore, considerable effort was given to evaluating rock net maintenance.

Input was solicited from maintenance personnel during all phases of this study. It was concluded that rock nets could be maintained within acceptable limits using standard maintenance equipment and procedures. In most cases, repairs and cleaning were completed in one to four hours.

It was found during the study that rockfall accumulations behind a single panel could be removed easily and quickly while still providing maximum protection to the workers and the traveling public.

Access for cleaning boulders and rockfall debris from the Brugg rock net and EI rock net was provided by raising individual panels from the bottom or by lowering them from the top. Both methods worked well.

In summary, cleaning and repair of a Brugg rock net restraining system and EI rock net restraining system can be accomplished by a typical maintenance crew using basic, readily available tools such as ratchet wrenches and sockets, torque wrenches, come-alongs, pry bars, and where possible, front end loaders.

CONCLUSIONS

- Design load rockfalls were effectively stopped by both rock nets.
- Repair and cleaning is required and can be done quickly and safely with equipment readily available at all maintenance stations.
- Modifications in design of both systems can be made to reduce repair.
- Brugg net panels deflected downslope as much as six feet under design load.
- EI net panels deflected downslope as much as 12 feet under design load.
- The EI net system requires more space than the Brugg system to accommodate downslope anchors.
- Chain link mesh is an integral part of the net design. The mesh prevents small rock fragments from passing through the net and reduces localized net damage.
- Connection of EI's net panel to the 5/8-inch-diameter perimeter wire rope was efficient and worked well.
- All of the perimeter wire ropes, guy wire ropes, and anchor wire ropes were properly sized and worked well.
- Brugg's friction brakes rarely activated. As a result, the energy which should have been dissipated by the brakes was transferred to weaker components.
- EI's friction brakes were effective in dissipating energy, but activated so easily that the nets sagged considerably even after a single design load impact.
- EI's support posts were damaged below design load impacts.
- Both foundation anchor designs provided adequate support to the net system.
- Anchor foundation locations can be offset two to three feet to accommodate difficult drilling conditions.
- Proper selection of a rock net requires a detailed site investigation.

RECOMMENDATIONS

The following recommendations are intended to serve as a guide to reduce maintenance on rock nets designed to contain 70 ft-tons of energy.

- Proper selection and design of rockfall mitigation measures should be based on a detailed site investigation.
- The Brugg 2.5 mm mild steel net fasteners are recommended for use with their system.
- Chain link fencing attached to the net panels is recommended.
- All attachments of the Brugg net panels should be made exclusively to the perimeter wire rope and adjacent panels rather than partially to the posts.
- When lacing is used to attach Brugg net panels, 5/16-inch wire rope lacing should be used.
- Brugg 3/4-inch friction brake tensile strength should be reduced to reduce repairs.
- Industrial Enterprise 5/8-inch friction brake tensile strength should be increased to reduce excessive sag.
- Industrial Enterprise support post strength should be increased to that of an W8 x 48 steel post to eliminate the need for post replacement.

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PERFORMANCE ANALYSIS IN ROCKFALL SIMULATION

by: Gordon M. Elliott[1]
Kenneth H. Rippere[1]

ABSTRACT

Rockfall simulation and mitigation have received considerable attention in recent years, owing to the balance between two sensitive issues. On the one hand Highway Departments seek to minimize their exposure to litigation and financial loss stemming from an unwanted rockfall event. On the other hand, Highway Departments are being asked to curb their spending when the cost of implementing 'absolutely safe' mitigation measures is increasing.

Predictions of rockfall behavior are uncertain, and there is always some, albeit small, chance of an unwanted event. It is misleading to believe that a mitigation measure, designed to prevent debris reaching the roadway with certainty on paper, will actually reduce rockfall hazards to zero. Performance analysis can help us quantify the relative chances of an unwanted event occurring for several alternative mitigation measures. With this information a Highway Department can participate more actively in selecting the most cost-effective measure.

This paper discusses the process of choosing a cost-effective rockfall mitigation measure, and discusses the role of rockfall simulation studies. The paper also comments on several aspects of rockfall behavior that are not adequately addressed by existing rockfall programs and suggests some areas for further research and development.

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[1] Golder Associates Inc., Atlanta, GA

INTRODUCTION

In the litigious society in which we now live, rockfall simulation and mitigation have received considerable attention, owing to the increasing financial and judicial consequences of rockfall debris reaching our highways. Increasing demand has been placed on engineers to develop 'absolutely safe' mitigation designs. However, absolute safety is an illusive goal in practice.

In practice, whether it be a rock cut, a tunnel, or any other structure, engineers regularly have to design in the face of uncertainty; uncertainty in the models to describe the physical setting and uncertainty in the values of the input variables to use in the analyses. Traditionally engineers develop designs using deterministic analyses, and attempt to ensure safety by:

- addressing 'worst case' scenarios;
- designing for prescribed factors of safety;
- carrying out sensitivity analyses (if budget allows); and
- including peer reviews.

The degree of conservatism in the design is often reflected in the construction cost. However, a deterministic analysis does not provide a definitive "yes" or "no" answer to the question "Will the design succeed?". The reason is that a deterministic analysis does not recognize uncertainty. Consequently, while the Engineer endeavors to design a "safe" structure, he can never be certain that it will be "absolutely safe", and thus "absolute safety is an illusive goal in practice. Instead, Highway Departments spend large sums of money on the basis of the Engineer's experience that the design should succeed given all the conservatism incorporated therein.

In the case of rockfalls, when faced with deciding between several different mitigation measures, each of which is unlikely to be 'absolutely safe' to highway users, decision analysts will assert that the best choice from several alternatives is that which maximizes the expected utility, and hence minimizes the risk. In other words, the best choice between several rockfall mitigation measures presents the most acceptable combination of:

- the confidence that, if a given measure is implemented, rockfall debris will not reach the highway with sufficient momentum to cause an accident; and

- the perceived value of, and willingness of the Highway Department to invest in, the mitigation measure.

The first of these items can be addressed using Performance Analysis.

Performance analysis focuses on trying to quantify the confidence in the design to achieve its objective. The Engineer and Highway Department first need to identify a benchmark outcome (e.g. momentum of a boulder at the guard rail), the magnitude of which can be calculated by the Engineer and the consequences of which can be translated by the Highway Department into tangible form (e.g. dollars). The performance analysis carried out by the Engineer quantifies the likelihood that the magnitude of the benchmark outcome will have a certain value. The Highway Department can then take that same likelihood to be the likelihood it will incur the consequences. In this way, a performance analysis provides a rational and defensible basis for the Engineer and the Highway Department to cooperate in choosing the most cost-effective alternative and hence share the risks. Therefore, a performance analysis has several important benefits to both the Engineer and the Highway Department:

- By agreeing to carry out a performance analysis, the relationship between the Engineer and the Highway Department is established on a rational basis and understanding that 'absolute safety' is a practical illusion;

- The Engineer is able to communicate to the Highway Department in quantitative terms his confidence in the likelihood of success of alternative designs;

- It enables the Highway Department to incorporate its perceived value of, and willingness to invest in, the project;

- It enables the cost-benefits of different designs to be compared on an equal basis; and thereby, allows the Highway Department to make the best decision; and

- It establishes a framework for discussing risk-sharing between the Engineer and the Highway Department through mutual understanding, and thereby diffuses potential future conflicts.

There are several drawbacks of not using performance analysis to assist in making the decision. Not only can a deterministic analysis can lead to an increase in the cost of construction projects as a consequence of the inherent conservatism, but also the lack of understanding of uncertainty in such an analysis sows the seed of potential conflict and litigation between the Engineer and the Highway Department in the future. The Highway Department, who pays for constructing the Engineer's design and will be the first to face the consequences of an unwanted event, is left without a satisfactory answer to his question "Will the design succeed?". Furthermore, the Highway Department is unable to participate in selecting the most cost-effective alternative because the Engineer has not been able to quantify the relative reliability of the alternative designs. The Highway Department, therefore, has to trust the Engineer to juggle reliability (safety) with cost-effectiveness (minimum risk for minimum cost) in the Highway Department's best interests. In this way the Highway Department loses some control over how he spends his money, the Engineer assumes greater liability for the final decision, and a false-impression is conveyed that Engineers can develop 'absolutely safe' designs. These are serious drawbacks to using deterministic analyses.

The procedure for carrying out a performance analysis is similar to a deterministic analysis, in so far as the Engineer still has to identify a suitable model that describes the physical setting, and the Engineer still has to collect and analyze data for each of the input variables. However, instead of choosing discrete values for each input variable and calculating a single outcome, a distribution of possible outcomes is calculated from distributions of the input variables by applying statistical sampling techniques to the models. In this way the uncertainties in the outcome are related directly to the site-specific uncertainties in all the known variables. Not only does this make the calculation site specific, but also the results can be directly integrated with the decision analysis to make the best choice of mitigation design to implement.

The distribution of each input variable is most easily derived by plotting all the data in the form of a histogram, choosing a mathematical expression, called a Probability Density Function (PDF), that most closely describes the shape of the histogram, and then analyzing the data statistically to define the constants in the PDF. In this way the uncertainties in the data for the input variables are quantified and can be incorporated in the Engineering calculation.

In carrying out a performance analysis, the Engineer is still obliged to select or develop appropriate models, and to collect and use the best information and data available. This is particularly apropos to applying performance analysis methods to rockfall simulation and mitigation. Therefore, the remainder of this paper provides a commentary on

- Rockfall models and reality; and
- typical distributions of input variables.

ROCKFALL MODELS

There are primarily three sources of rockfall debris:

- Discrete boulders which are dislodged high up on a slope and roll/bounce down the slope;

- Discrete boulders which shatter on impact and break up into small pieces that are projected off with much higher velocity than the parent boulder; and

- Localized slope failures which lead to a shower of boulders that avalanches down the slope.

These sources, and rockfall processes are described separately.

Discrete Rockfalls

Many workers, both within the U.S. and throughout the world, have proposed models for predicting discrete rockfall behavior in recent years (Benitez et al., 1977 [Spain]; Piteau and Associates, 1980 [Canada]; Wu, 1984 [U.S.A.]; Chan, et al., 1986 [Hong Kong]; Descoeudres and Zimmermann, 1987 [Switzerland]; Spang, 1987 [Germany]; Paronuzzi, 1989 [Italy]; and Pfeiffer & Bowen, 1989 [U.S.A.]). All of these discrete rockfall simulation models have been developed using Newtonian mechanics. All of these workers have recognized that rockfall prediction is uncertain because of the random way a boulder rebounds after impact, and consequently each model has some method of generating a distribution of calculated results. However, most of the existing models have been developed for specific applications, and therefore, few are able to generate results that are consistent with all observations of rockfall behavior.

Some models do not incorporate sliding or roll characteristics of rockfall behavior and focus only on bounce characteristics. Furthermore, some of the models are unable to reproduce the size effects often observed in practice because they do not relate slope roughness to boulder size. In the absence of this relationship, travel distance calculated on the basis of discrete particles rolling or bouncing will be independent of particle size.

Ritchie (1963) commented that shape of a boulder has little importance on its falling or rolling characteristics on inclined slopes unless the boulder is long, like a pencil. In the case of near spherical boulders, irregularities in shape can be accounted for in the levels of uncertainty in other variables, such as surface roughness or rolling friction. In the case of tabular or oblong boulders, if the boulder stays close to the ground, the irregularity of shape retards roll and consequently, calculations that assume a spherical boulder will overestimate the distance travelled. However, if the irregularly shaped boulder bounces clear of the ground surface between impacts, the shape can contribute significantly to the randomness of rockfall behavior, as observed recently by the authors.

In a recent field test, the authors pushed oblong and tabular boulders off the crest of a 70-foot to 100-foot high vertical cliff. These boulders fell onto a smooth horizontal compacted earth fill surface, and travelled out from the toe of the cliff as far as 50 feet to 60 feet. This result cannot be simulated using traditional approaches to rockfall simulation, and is assumed to be the result of the increase in angular velocity of the boulder during the impact. This increase in angular velocity needs to be determined and then added to the impact velocity components before the rebound velocities are calculated. This is a form of rockfall behavior that is not accounted for in existing rockfall models.

Another important feature of simulation models that will affect the ability to predict total distance travelled is the termination criteria used, both to define the end of one travel mode and the start of another (e.g. change from bouncing to rolling), and to define the end of the simulation. Few of the existing models clearly state the termination criteria, and of those that do, none enable the user to assign the value for the variable that is used in the termination criteria.

Engineers are trained to work with simple models of the physical setting. Generally, useful results can be obtained without the need to expend considerable resources on further refinement. However, even simple models are worthless if they are incorrectly applied. Each of the items discussed above will affect the likelihood of the outcome of a rockfall event. The absence of algorithms in a model to account for these behavior characteristics could lead to the model misrepresenting the likelihood of an outcome of a given magnitude. In view of this discussion, the authors are in the process of developing their own rockfall model and hope to make it available in the next 12 months.

Shattering Rocks

Occasionally, through the combination of several 'ideal' conditions, a boulder which shatters on impact with a hard surface can project smaller fragments at high velocity in random directions. A deterministic analysis of this event could lead to an extraordinarily conservative design for a rockfall mitigation structure.

Performance analysis techniques could reconstruct his event and provide a distribution for the momentum of the projectiles reaching the highway, given that the event does occur. However, in practice the probability of such an occurrence is not only small, but also difficult to quantify. Invariably a subjective assessment has to be made. One may find that the product of the probability of the event occurring and the probability of a projectile having sufficient momentum to cause an accident on the highway, given that the event occurs, is so small that the cost to mitigate against such an event is unjustifiable.

Rock Slides

Almost exclusively, rockfall models available today investigate the behavior of a discrete boulder as it travels down a slope and comes to rest on a surface below the point of release. Perhaps a more common occurrence is a rock slide, in which a portion of the rock mass becomes detached and, in breaking up, cascades down the slope. Events of this type have not been analytically studied but experience permits a few pertinent observations. It is our experience that a well maintained ditch which has been designed to contain the discrete boulder will also contain rock slides unless the slide has sufficient volume to fill the ditch and overflow into the highway. Rockslide materials appear to reach the toe area with considerably less energy than individual boulders. One might speculate that much energy is dissipated in the interaction of cascading rock particles. In addition, the configuration of the toe of the slope is different for each discrete boulder, as other boulders are brought to rest.

In terms of a performance analysis of this situation, one could combine the probability of the expected slide volume (and consequently size of debris pile at the toe of the slope) with the conditional probability distribution for a discrete analysis of the last boulder to come down the slope. This boulder will see a different geometry at the base of the slope and different material property values than the original cut profile. Rockfall mitigation measures can then be designed to reflect likely conditions.

DISTRIBUTIONS OF VARIABLES GOVERNING ROCKFALL BEHAVIOR

Most of the rockfall models cited in the reference list acknowledge that the values for the input variables have to be defined in the form of distributions. However, few of these model either combine the uncertainties of all the input variables simultaneously, or use distributions of parameter values that reflect those observed through direct field measurement. Some important variables that influence rockfall behavior are slope roughness, coefficients of restitution, and boulder size. Distributions for each of these variables are discussed separately.

Slope Roughness

The slope geometry is most easily represented by several straight line segments. Each straight line segment has an average inclination and an associated slope roughness. Slope roughness as referred to herein is the difference in angle between the inclination of the plane of impact and the inclination of a straight segment of the slope, as illustrated in Figure 1. Slope roughness is likely to be normally distributed. Certainly, data collected by the authors for a rock cut in New Jersey were normally distributed as illustrated in Figure 2.

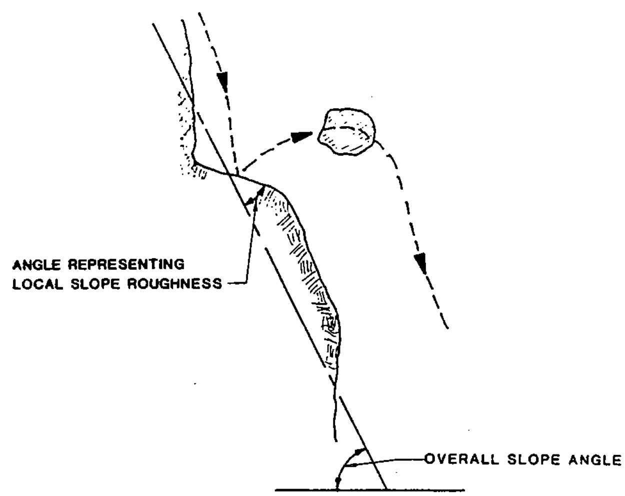

FIGURE 1: ILLUSTRATION OF SLOPE ROUGHNESS

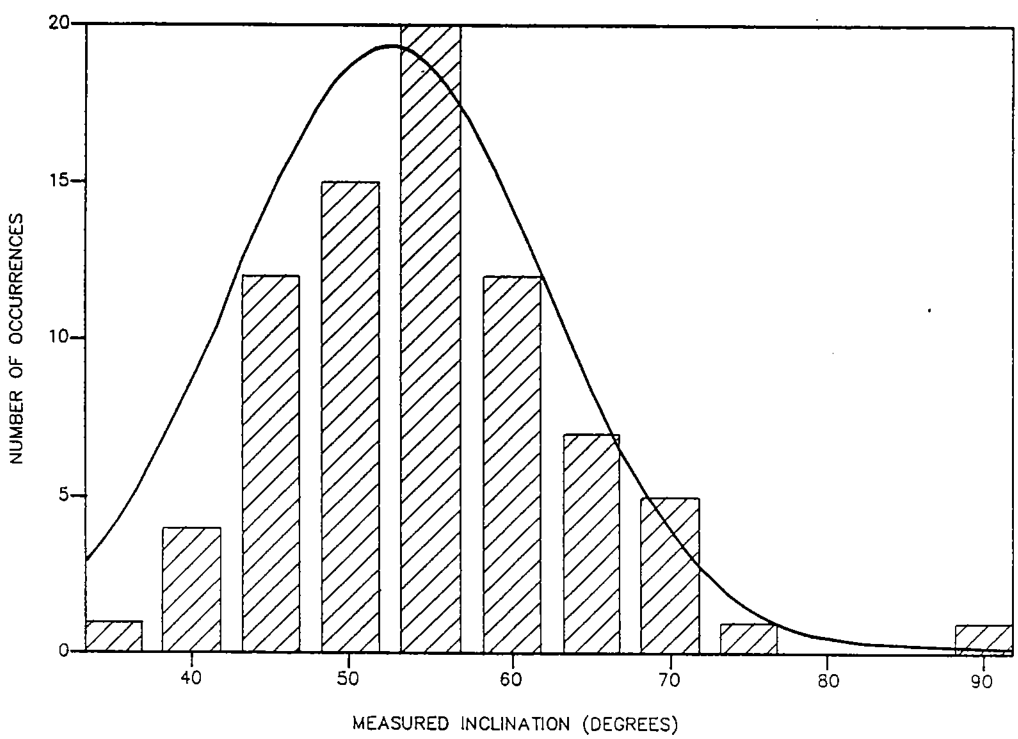

FIGURE 2: DISTRIBUTION OF SLOPE INCLINATION MEASURED USING AN 18-INCH ROD

These data were collected using the simple straight edge and dip-meter method suggested by Fecker and Rengers (1971). The mean of the slope roughness distribution is zero, and the standard deviation is equal to the standard deviation of the slope inclination data. One important observation is that the inclination of the impact surface can be steeper or flatter than the overall slope inclination.If data are collected using several straight edges of different lengths, measurements of apparent slope roughness for boulders of different size can be made. A graph of the calculated standard deviation plotted against straight edge length is presented in Figure 3 for the same New Jersey rock cut. This result confirms that the variation of slope roughness that will be experienced by a small boulder is greater than that experienced by a large boulder.

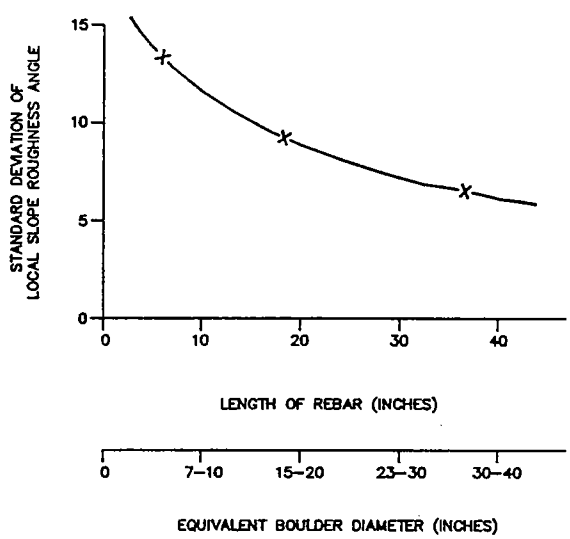

FIGURE 3: RELATIONSHIP BETWEEN BOULDER SIZE & SLOPE ROUGHNESS

Slope roughness should not be confused with changes in geometry. A ledge formed by collaring out pre-split holes in blasting a high rock cut is not a slope roughness feature if the width of the ledge is larger than the diameter of the boulder being assessed and it must be treated in the same way as a bench. It should also be noted in passing that slope roughness is not just dependent on the distribution of fractures and discontinuities in the rock mass. The method of construction, and particularly the quality of blasting, is also an important influence.

Coefficients of Restitution

The coefficients of restitution in directions normal and tangential to the plane of impact govern the ratio of the rebound to impact velocities normal and tangential to the plane of impact respectively. It might also be assumed that data for these coefficients are normally distributed. However, these coefficients can only have values between zero and one. This presents limitations on the degree of uncertainty that can be assigned, particularly when the value of the coefficient approaches the limits, such as for very hard or very soft surfaces. Under these circumstances it is suggested that a beta distribution or any other skewed distribution might fit the measured data better. This hypothesis has also been suggested by Paronuzzi (1989). However, field data are needed to provide validation. Some illustrations of shapes of beta distributions are presented in Figure 4.

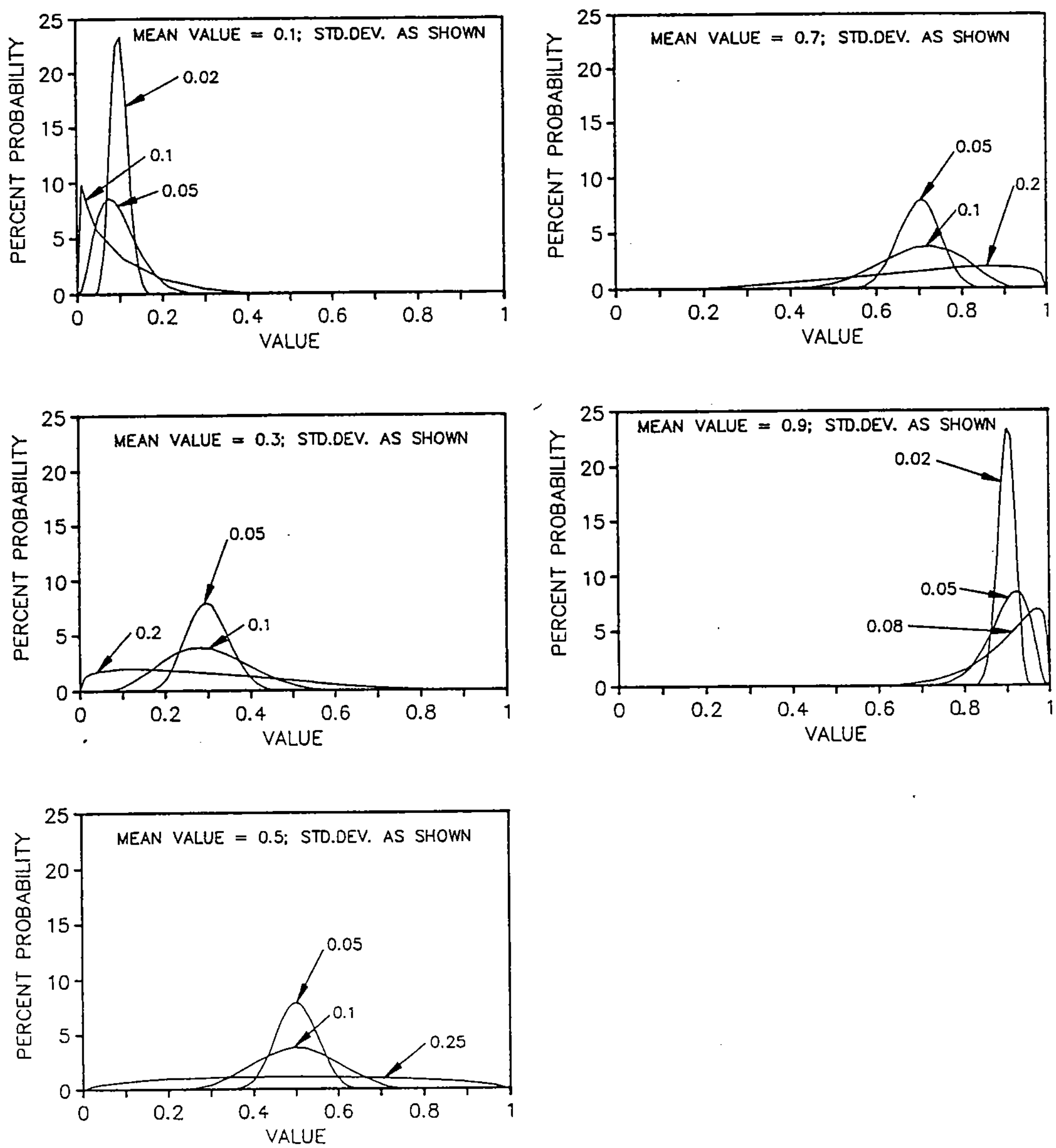

FIGURE 4: EXAMPLES OF BETA DISTRIBUTIONS

The coefficient of normal restitution can be measured in the field using a simple vertical drop test. However, the coefficient of tangential restitution is difficult to measure directly in the field. One indirect method, first suggested by Wu (1984), is a variation of the drop test. This involves dropping a boulder from a height, onto an inclined plane of known dip angle, and measuring the distance down slope between the first and second impacts. However the relationship between the coefficient of tangential restitution and the distance between impacts is complex, and requires that the coefficient of normal restitution be known. Since the latter is itself a distribution, this uncertainty will be included in the uncertainty in the coefficient of tangential restitution.

Coefficient of Rolling Friction

The coefficient of rolling friction governs the rate of rotational deceleration of a rolling boulder. It has a value between zero and unity, and therefore might also be fitted by a beta distribution or other skewed distribution.

A near spherical boulder will only roll down an inclined surface if the moment of its weight about the point of contact with the plane exceeds the restraining moment of rolling friction about the centroid. The tangent of the inclination of the plane down which a boulder just fails to roll is equal to the coefficient of rolling friction. This clearly lends itself to a simple field test procedure.

Those in the mining industry who have observed boulder roll-out from the sides of mine waste piles know that the coefficient of rolling friction is generally greater than $\tan^{-1} 23^{o}$. This value takes into account the angularity of rolling boulders, and is clearly too high for purely spherical boulders. However, by choosing a value measured in this way, an analysis based on spherical boulders will tend to reproduce the behavior of irregularly, but near spherically, shaped boulders.

It is hypothesized that the roll-test can also be used as an alternative approach to determining the coefficient of tangential restitution if it is assumed that it is related to the coefficient of rolling friction. The following relationship between the coefficient of tangential friction, Rt, and the coefficient of rolling friction, F,

$$Rt = \frac{1}{1 + F} \qquad (1)$$

satisfies energy considerations of both bouncing and rolling spheres. However, this hypothesis has not been validated to date, and requires investigation through field measurement. If validated, this approach would have a significant advantage over the drop test approach of Wu (1984) in that the coefficient of tangential restitution can be measured directly using a simple roll test.

Boulder Size

An obvious and important variable in a rockfall analysis is boulder size. This variable influences both the potential consequences of a rockfall (a large boulder in practice tends to cause more damage than a small boulder) and the design of any rockfall retention structures. It is therefore advisable in any field investigation of a potential rockfall hazard to carry out a survey of boulder size in the source areas.

Boulder size can be included in the performance analysis in two ways. One is simply to use the boulder size survey data to identify a finite number of sizes to analyze, and then carry out a finite number of analyses, one for each size, in order to identify which boulder size is most critical. However, although this approach is helpful in identifying which boulder size is most critical, it does not necessarily provide the information needed to make the best choice from several alternative remedial measures.

An alternative approach is to include the boulder size distribution (e.g. a log-normal distribution) in the performance analysis, and write the program to output both the distribution of the boulder sizes passing a given point on the slope and the distribution of the momentum of boulders passing that point. This approach cannot be easily used to differentiate between boulder sizes in the output. However it is more useful to the designer of a rockfall retention structure because it quantifies the chance that the structure will have to withstand a given impact force. It is also more useful to the Highway Department since the consequences of a boulder reaching the highway are more likely to be related to the momentum rather than any other variable.

Conclusion

Despite the publication of numerous models in recent years, rockfall is still a major issue lacking definitive answers. In the real world, it is not always cost-effective to design a rockfall debris retention measure that ensures that no rock will ever reach the highway and cause an accident. Highway Departments who own property with rockfall hazards must recognize that Engineers cannot provide comprehensive cost-effective designs that are 'absolutely safe' because 'absolute safety' is an illusive goal in practice. There is always a chance, albeit small, that a boulder might bounce over/through a retention structure, or will shatter on impact.

Unlike conventional deterministic approaches, performance analysis approaches can be used to relate the relative likelihood of an unwanted rockfall event to the uncertainties in the variables used to simulate rockfall behavior. Consequently, information can be generated to help select the rockfall mitigation measure that is likely to be most cost-effective at minimizing risk. Performance analysis has important benefits to the Engineer and the Highway Department alike. Above all it enables the Highway Department to participate actively in choosing the most cost-effective alternative, and it reduces the Engineer's liability for the design selection.

The Highway Department should be prepared to share risk and either define acceptable levels of chance of an unwanted event, or relate the magnitude of an outcome to tangible consequences. To this end, a good rockfall simulation program based on performance analysis concepts and site specific PDFs provides a useful design tool for use in conjunction with cooperative risk-based decision making.

It is evident that there is still scope for further research of rockfall behavior and improvement of rockfall models. The authors have started developing a rockfall program that addresses the issues discussed herein, and have started collecting and compiling data for validating the results. The authors hope to be able to report progress with program development at the next Highways Symposium.

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DESIGN OF GEOGRID WALL WITH WICK DRAINS IN TUCUMCARI, NEW MEXICO

by

Edward Rector, P.E.
Richard Lueck, P.E.
Materials Laboratory Bureau
New Mexico State Highway & Transportation Department

Abstract

The Tucumcari Mountain Road Extension project extends existing Mountain Road in east Tucumcari, New Mexico, from I-40 to US 54. This alignment required construction of two 40 foot high embankments to cross the Southern Pacific Railway. Soft, wet, fine grained soils in the project area west of Tucumcari Lake, necessitated construction of two 40 foot high temporary, embankments reinforced with geotextiles. Wick drains with internal drainage appurtenances were also installed in the foundation soils to accelerate the consolidation of the existing foundation soils before the permanent bridge abutments were to be placed. The anticipated time to 90% consolidation was 120 days. The temporary embankments were instrumented to record the actual settlements and actual pore pressures in the foundation soils.

Introduction

The purpose of this report is to discuss the consolidation settlement, slope stability analyses, construction considerations, and long term water table effects for construction of a 2.2 mile long two lane roadway extension of Mountain Road Bypass in Tucumcari, New Mexico. This includes two 40-foot high bridge abutment approach embankments. The entire roadway overlies approximately 130 to 150 feet of interbedded saturated silts, clays, and sandy silts. The site has a high water table and surface water run-off causing local "boggy" areas. The report includes the site investigation, soils encountered, lab and field tests performed, consolidation predictions, alternates considered for consolidation acceleration, wick drain design and layout, instrumentation and monitoring of the foundation during the fill construction.

Physiographic Setting

The project area is located within the Tucumcari Basin province. It lies north of the Llano Estacado, southeast of the Canadian River Escarpment and southwest of the Great Plains Province. The proposed alignment for the Tucumcari by-pass route will skirt the western shore of Tucumcari Lake.

Geology

Tucumcari Lake is a natural lake. There are two hypotheses which explain the formation of the lake basin: (1) the Tucumcari drainage previously flowed north to join Pajarito Creek but was impounded during the Quaternary Period by sand dunes up to 60 feet high (Dobrovolny and

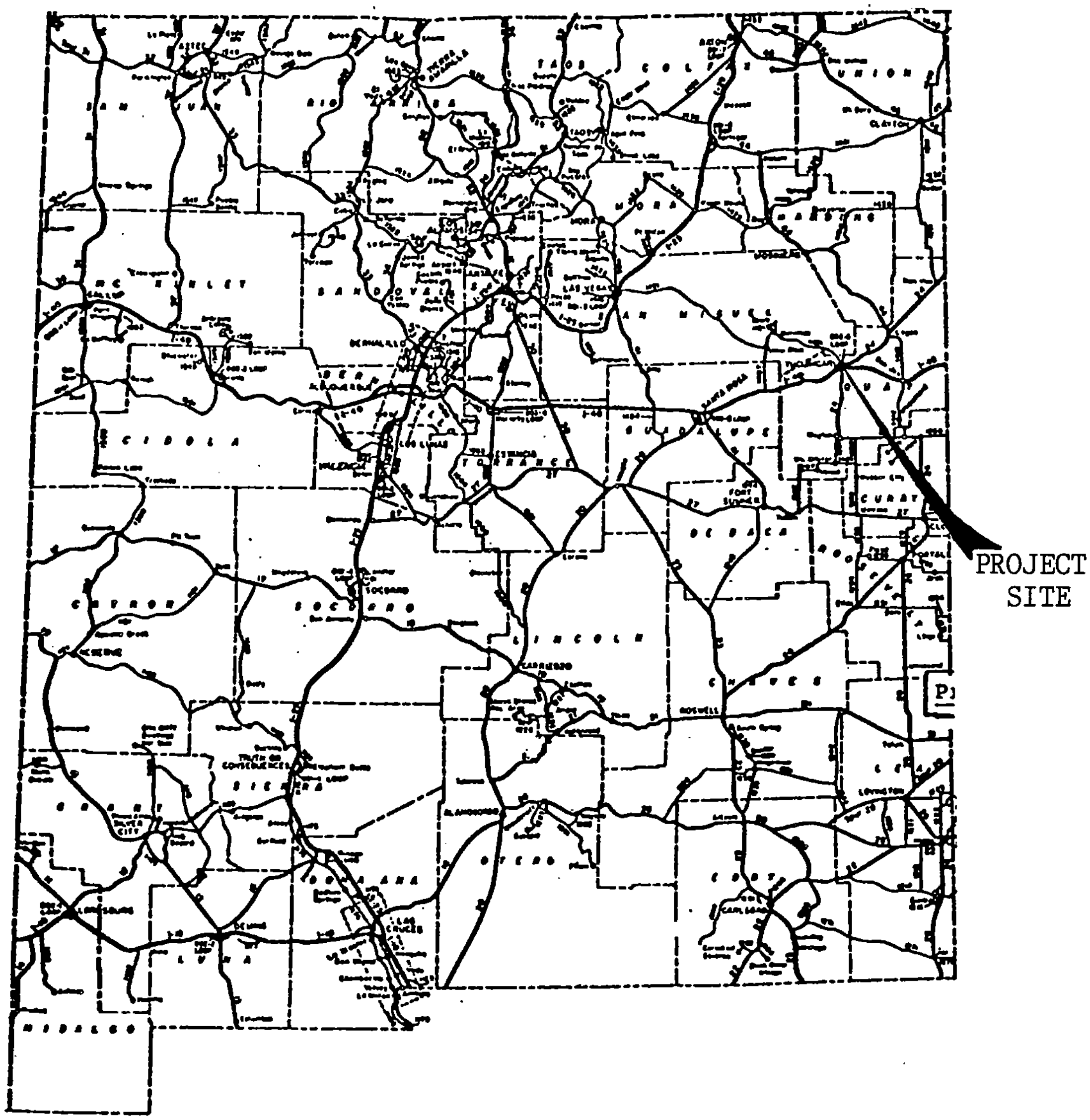

FIG. 1
VINCINITY MAP

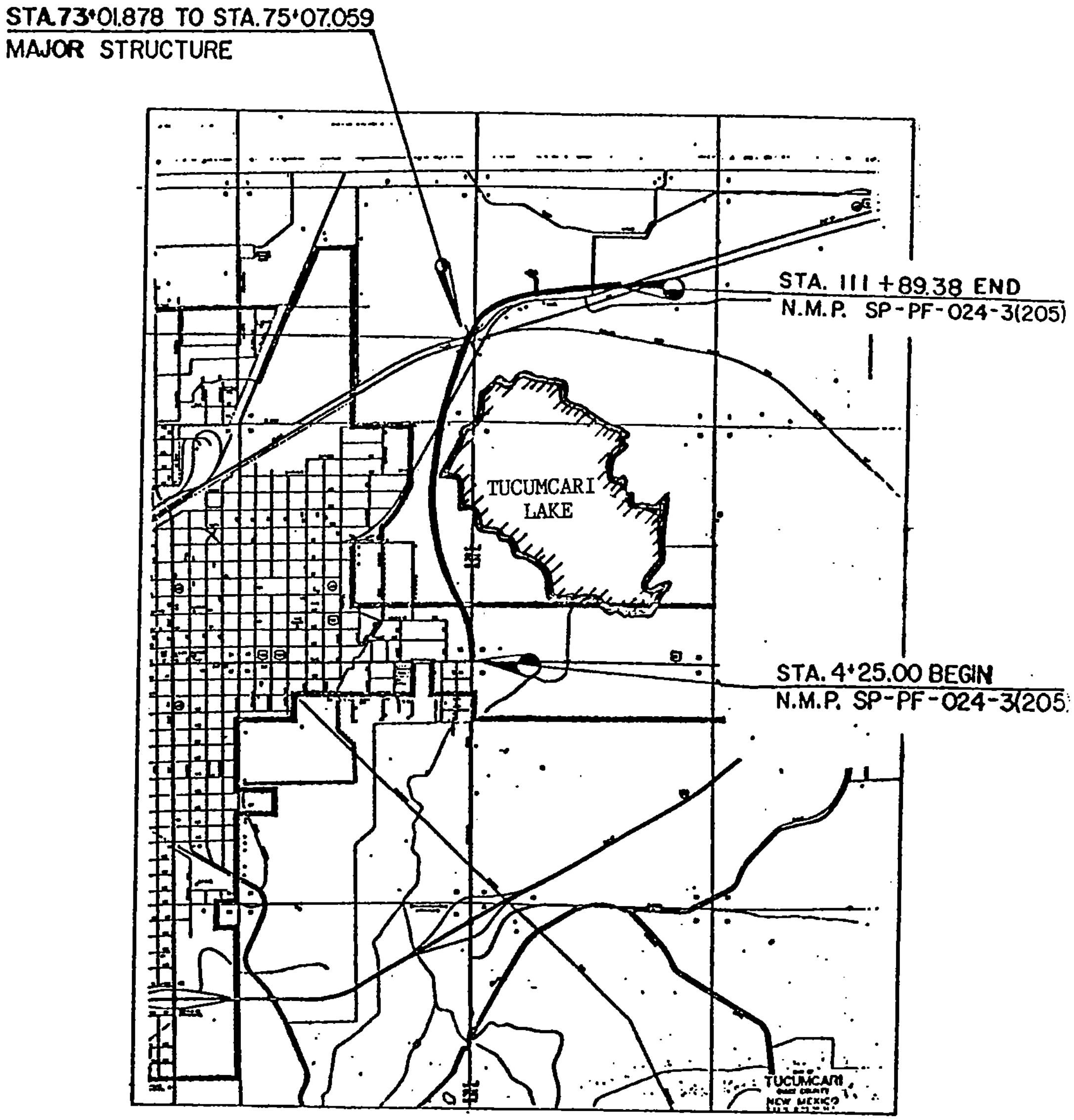

FIG. 2
PROJECT LOCATION

FOUNDATION SOIL PROPERTIES

HOLE #	DEPTH (ft.)	DRY DENSITY (pcf)	MOISTURE %	SOIL TYPE	q (psi)	TRIAX(UU) C(psi)	Ø	DIRECT SHEAR C(psi)	Ø	Field Cc	e_o	G
1	4 – 6	103.3	18.6	ML	–	–	–	–	–	0.185	0.7240	2.686
3	25	110.9	19.2	CL	6.3	–	–	–	–	–	–	–
4	5	100.3	24.0	SM	–	–	–	0	33.5	–	–	–
4	40	117.7	12.7	ML–SM	–	3.0	20.5	–	–	–	–	–
5	10	104.2	21.5	CL	–	–	–	–	20.0	–	–	–
5	30	91.6	29.8	CL–CH	6.4	–	–	–	–	0.370	0.8180	2.528
8	5	108.9	10.3	SM	2.3	–	–	–	–	–	–	–
8	40	102.8	22.0	ML	–	–	–	–	–	0.142	0.6250	2.588
9	15	105.6	18.5	CL	–	–	–	–	–	0.100	0.3430	2.566
9	25	111.9	16.8	CL	–	–	–	–	–	–	–	–
10	35	107.3	21.4	CL	–	–	–	–	–	–	–	–
11	20	108.7	17.0	SM	4.5	–	–	–	–	–	–	–
12	20	103.7	18.9	ML	–	1.0	30.0	–	–	–	–	–
13	15	103.5	22.0	SM	5.9	–	–	–	–	–	–	–
15	40	120.5	16.3	CL	–	0.0	35.0	–	–	0.154	0.4163	2.735

TABLE 1

FOUNDATION SOIL PROPERTIES

HOLE #	DEPTH (ft.)	DRY DENSITY (pcf)	MOISTURE %	SOIL TYPE	q (psi)	TRIAX(UU) C(psi)	Ø	DIRECT SHEAR C(psi)	Ø	Field Cc	e_o	G
2A	10	-	18.0	ML	-	-	-	-	-	-	-	-
3A	10	100.8	27.0	CL	-	4.3	33.5	-	-	-	-	-
4A	15	-	18.6	ML	-	-	-	-	-	0.140	0.4163	2.603
4A	25	-	14.7	CL	-	-	-	-	-	0.120	0.4895	2.708
4A	60	115.6	19.4	SM	-	1.0	19.5	-	-	-	-	-
4A	70	109.2	22.3	SM	-	-	-	-	-	-	0.5216	2.672
4A	80	112.2	23.1	SM	-	-	-	-	-	-	0.4778	2.656
A1	110	117.7	3.0	ML	14.9	-	-	-	-	-	-	-
A1	125	115.4	14.6	ML	-	-	-	-	-	0.118	0.4806	2.697
A1	130	-	16.5	ML	-	-	-	-	-	0.240	0.6125	2.706

TABLE 1 (Continued)

Townsend, 1946) or (2) the Tucumcari Lake basin was formed by solution subsidence in the underlying Permian Age evaporites (Trigger and Bushman, 1964).

Stratigraphy

The stratigraphic units in this area include Quaternary age (recent to 1 million years) alluvium, and pediment deposits. Bedrock in this area is the Chinle Formation which is Triassic in age (11 m.y. to 70 m.y.). The alluvium consists of interbedded sand, silt and clay. The pediment includes interbedded sand, and silt, with some clay and fine gravel. The Chinle Formation includes red and brown siltstones, sandstones and variegated shales.

Soils

The soils in the project area were derived from the surrounding Chinle Formation (Triassic). This formation consists of siltstones, mudstones, shales, and sediments characteristic of deep, quiet, marine environments. Regionally, Tucumcari may lay at the center of a structural basin. Tucumcari Lake, which is situated east of this project, has no major inlets and is recharged with surface runoff and subsurface flow. The soil thickness in the overpass area appears to be in the neighborhood of 130 to 150 feet, as determined by one boring of 200′ total depth.

Site Investigations

The subsurface investigation for the bridge site and roadway was conducted in June, 1988. A total of 20 hollow stem auger (HSA) borings, 4 Dutch Cone Penetrometer soundings, and 1 boring to bedrock were performed at various locations along the roadway and at the bridge overpass site. Typically, the HSA borings were placed near drainage structure locations along the roadway. Four borings were placed at or near the abutment and pier locations for the bridge overpass structure. The deep bedrock boring was done near the bridge site where both spontaneous potential and resistivity logs were obtained. Standard Penetration Tests (SPT) were performed in the HSA borings. Disturbed samples were obtained from this test by driving a 2 inch O.D. split barrel sampler a total of 18 inches by using a 140 pound hammer dropping a distance of 30 inches. The number of blows required to drive the sampler the final 12 inches was recorded as the 'N' value. Relatively 'undisturbed' 3 inch I.D. thin wall tube samples were also obtained through the HSA borings.

The Dutch Cone Penetrometer tests were performed by pushing both the conical point and friction sleeve with the hydraulically operated kelly of the MOBILE DRILL RIG. THE penetrometer was advanced at a constant rate of 0.30 inches per minute. Both the cone and cone plus friction sleeve resistances were read and recorded. A computer program was used to reduce the data and produce graphs of depth versus cone resistance, sleeve resistance and friction ratio.

Both natural gamma and electrical geophysical logs were obtained near one abutment of the bridge overpass. The natural gamma log is a measure of the natural gamma radiation in a soil mass. Theoretically, the fine grained material will show higher radiation than the coarse grained material. The electrical geophysical logs include the spontaneous potential and the resistivity. The spontaneous potential is a measure of the natural electrical currents that occur in soil surrounding the bore hole. The theory behind the use of this instrument is that when permeable zones are invaded by filtrates from ground water, a current will flow. Clays have extremely low permeability, whereas sands have a substantially higher degree of permeability. The resistivity log is a measure of the soil surrounding the bore hole resistance to an induced current. Clays will display a low resistivity, while sands and gravels will have high resistivity.

Laboratory Tests

Lab tests were performed on both the disturbed split spoon samples and the 'undisturbed' thin wall samples.

Gradations, Atterberg limits, and moisture determinations were obtained from the split spoon samples. The 'undisturbed' samples had gradations, Atterberg limits, moisture, fixed ring consolidation tests, direct shear (C.D.) tests, triaxial shear tests (U.U.) and unconfined compression tests performed on them.

The gradations and Atterberg limits were used to classify the soils using both the Unified and AASHTO Classifications Systems. The consolidation test results were used to determine the time-rate of consolidation settlement and the amount of both primary and secondary settlement. The shear test results were used for both short and long term shear failure, slope stability analysis, and for bearing capacity failure analysis.

Foundation Conditions

The twenty borings show a large variety in subsurface conditions from boring to boring. There are no easily traceable soil layers from boring to boring. While each boring displays similar soil types - clays, silts and silty sands, the thickness, depths and order in which each soil type is encountered vary significantly. This is surprising since the local surface geology does not vary appreciably.

Table 1 summarizes the field and lab test results for the HSA borings along the roadway alignment, at the bridge abutments and piers.

Consolidation Analysis

A total of nine fixed-ring consolidation tests were performed on the saturated clays and silts. The samples ranged in depth from five to one hundred thirty feet. The C_c values ranged from .12 to .24 for the silt (ML) with an average of .165, and ranged from .10 to .370 for the clay material (CL, CH) with an average of .186. These values compared favorably with various empirical equations from Terzaghi and Peck, (C_c=0.009(LL-10)) and/or from Holtz and Kovacs, (C_c=0.75(e_o-.50)); (C_c=0.007(LL-10)). (See Table 2.)

Boring	Depth	Soil	Empirical Equ.	Empirical Cc	Field Cc
1	5	ML	$0.75(e_o-.50)$	.167	.185
5	30	CL	0.009(LL-10)	.306	.373
8	40	ML	$0.75\ (e_o-.50)$	.095	.142
9	15	CL	0.009 (LL-10)	.126	.100
15	40	CL	0.009 (LL-10)	.117	.154
4A	15	ML	$0.30\ (e_o-.27)$	.044	.14
4A	25	CL	0.009 (LL-10)	.144	.12
A1	125	ML	--	--	.12
A1	130	ML	$0.75\ (e_o-.50)$	.084	.24

TABLE 2

All samples tested for consolidation showed some effect of preconsolidation. The overconsolidation ratios (OCR), which is the ratio of past preconsolidation pressures divided by the present overburden pressures, for the samples tested ranged from 1.6 to 8.4 with an average of 3.3. In researching the geologic literature, there was no justifiable way to account for the higher overburden pressures and/or effective stresses. Consequently, the high OCR's were attributed to sample disturbance during drilling exploration activities. To consider the soils normally consolidated, in light of no supporting geologic evidence, is the most conservative approach, since total consolidation settlements will be significantly higher than those of a preconsolidated nature.

A consolidation analysis was performed for several embankment heights and configurations: A 5 foot embankment height models the settlement in the foundation material from Station 4+00 to Station 50+00, and a 40 foot embankment height models the settlement of the foundation material at the bridge overpass at the abutments.

The compressible soils at each of the locations were determined by the drill hole borings dispersed along the alignment. The following average values of compression index, Cc, were used for the calculations:

$$C_c = 0.15,\ \text{ML}$$
$$C_c = 0.12,\ \text{CL, CH}$$

The change in effective stress for each embankment configuration was determined by various computer and manual (influence charts) methods.

Two important conditions must be addressed with construction of the embankment in this area: consolidation settlements, both total and differential; and the stability of the foundation and embankment material in this high water table setting.

A coefficient of consolidation, CV, of .005 in^2/min. was used for the CL material and a .010 in^2/min value was used for the ML material to determine the time rate of consolidation without wick drains. Because horizontal permeability is generally greater in soils than vertical permeability, the higher coefficient of consolidation of .010 in^2/min. was used for the wick drain analysis.

Immediate settlement in the silty sand material in the 40 foot embankment area was found to be less than 0.1 ft. Immediate settlement in the clay and silt soils are 0.4 ft.

Secondary consolidation settlement in the clay soils in the 40 foot embankment was calculated to be less than 0.01 ft. after 20 years.

The time to 90% consolidation settlement without wick drains or surcharge loads in the 40 foot embankment area is shown in Table 3.

	Abutment #1	Abutment #2
Upper 75′	90 days (6.9")	350 days (10.8")
Lower 57′	900 day (6.9")	420 days (5.3")
Total 132′	660 days (13.9")	400 days (16.1")

Table 3

Because primary consolidation settlements and differential settlements were expected to be large and the time-rate of settlement relatively long, special construction sequences and techniques were considered along the roadway embankment, at major drainage structures, and at the bridge overpass abutments, to address these problems.

The borings for abutment #1 show that construction at this abutment could be accomplished without wick drains since settlement in the upper 75 feet would take place in a suitable time frame, but since the borings throughout the roadway alignment show a great degree of variability in the soils we recommended that both abutments receive wick drain and surcharge treatments.

The water table is only very high in the low embankment areas. In these areas it is about five feet below the natural ground surface. In this area, the water level could temporarily rise a slight amount (one foot). This estimate is based on the typical or average decrease in void ratio shown by the increase in effective stress of the consolidation curves of the foundation soils. This temporary rise would eventually subside to a level close to the preconstruction level.

Wick Drain Design

The success of wick drains used in acceleration of consolidation settlement for another project in Mescalero, New Mexico, encouraged the use of wick drains of this project.

The wick drains considered for this project have a corrugated or 'nubbed' plastic core that is encased in a geotextile filter fabric. The cross-sectional dimensions are approximately 4 inches wide and .25 inches thick. The drain is usually supplied on 100 foot rolls. The wick is installed by threading it through a hollow steel 2"x6" (O.D.) mandrel, stapling a metal galvanized steel plate to a fold of wick extended through the base of the mandrel, and vibrating or pushing the mandrel into the ground. When the mandrel reaches a predetermined depth or refusal, it is withdrawn from the ground. As the soil mass closes around the wick, the wick is held in place, and remains installed in the foundation soils as the mandrel is removed.

The wick drain design is based on the Barron-Kjellman equation modified for smear, the same equation that is used to design sand drains. Vertical drainage is ignored in the calculations and only radial drainage due to the wicks is considered.

Both U% and the wick drain triangular center spacing were varied to obtain different values of t. Table 3 illustrates some of the values obtained for this project based on a coefficient of consolidation of 0.005 $(In)^2$/Min.

	IN OVERPASS AREA	
Spacing (ft)	t (days) for U = 50%	t (days) for U = 90%
5.0	39	130
6.0	62	208

Table 4

The actual settlement times would be shorter because vertical drainage was not considered.

A wick spacing of 5 feet was chosen at the abutments for this project to give an acceptable time frame for consolidation settlement.

The choice of acceptable wick drains and the wick specifications were based on the experiences of New Mexico, California and Illinois.

Considerations should be based on the wick's flow capacity, the required pressure, the filtration ability, and the probability of the wick becoming blocked with the surrounding soil. CALTRANS developed a test procedure that can be used to evaluate different wick drains.

Based on this procedure, the following five wick drains were considered acceptable for the soils at Tucumcari.

1) Castle Board Drain
2) Mebra Prop Drain
3) Hitek
4) Alidrain
5) Amerdrain

Table 5 Lists Drainage Structures which are recommended to receive wick drain foundation soil treatment.

Station	Size	Wick Drain Treatment
73+20	120" x 286'	Yes (w/overpass)
78+00	96" x 260'	Yes

TABLE 5

Slope Stability

Embankments on this project will range from a minimum fill height of 5 foot to a maximum of 40 foot at the railroad overpass. Slopes were analyzed by the STABL4M computer program from Purdue University, for short and long term stability using Bishop's Modified Method. The fill characteristics used are: $c=0$, $\varnothing=35^{\circ}$, $\gamma=140$ pcf. Short term characteristics of the existing foundation material are: $c=450$ psf, $\varnothing=0$, $\gamma=125$ pcf. Long term characteristics of the existing material are: $c=0$, $\varnothing=20^{\circ}$, $\gamma=125$ pcf. Since the height of the embankment is considerable in the overpass area, different fill heights were examined for short term stability in relationship to pore pressure build-up in the foundation soils. Figure 7 shows the height of embankment vs. pore pressure in the foundation soil to a depth of approximately 25 feet. During construction of the embankment it is important not to exceed these pore pressures.

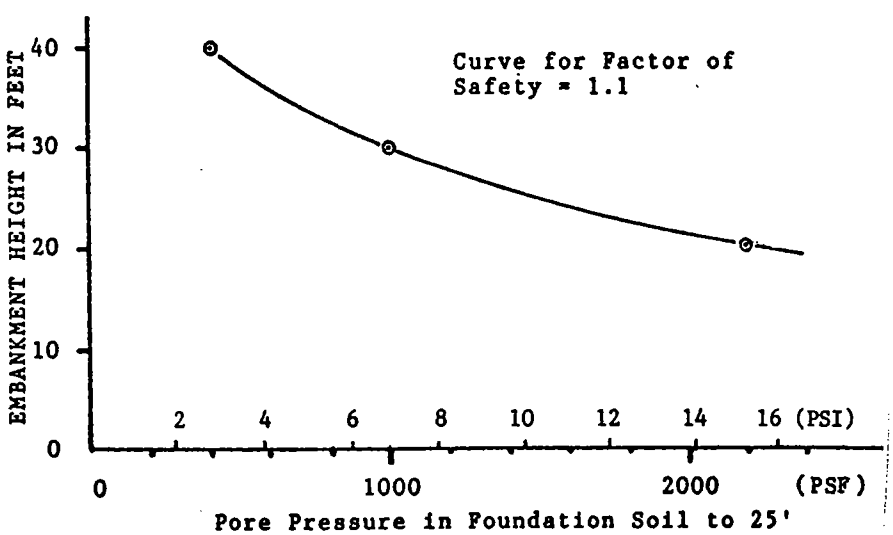

FIG. 7

Long term stability of the 40 foot embankment is estimated to have a Factor of Safety of 1.78.

Short term stability of the temporary wall is found to have a F.S. = 1.26. Pore pressures in the high fill and the temporary wall areas need to be monitored during fill construction and compared with Figure 7 to insure that the pore pressures generated in the foundation soils do not exceed those shown in Figure 7 in order to prevent foundation shear failure.

Bearing capacity of the foundation soil beneath the temporary wall was evaluated and found to be acceptable assuming the temporary wall is constructed slowly enough.

Instrumentation

Geotechnical instrumentation was installed at the temporary walls and high embankments at the bridge abutments and at all fill surcharge locations where drainage structures were installed. Instrumentation was completed immediately after the wick drain installation at the wick drain sites and after ground leveling at the pipe structure surcharge locations. The instrumentation was used to monitor possible foundation shear failure, time rate of settlement, and pore pressure build-up and dissipation. The types of instrumentation included slope inclinometers, settlement transducers and pore pressure transducers. The instrumentation be placed at the the following locations:

Structure	Instrumentation Type	Instrumentation Location
Temporary bridge abutments	Inclinometers (2 each)	At centerline of roadway about 10 feet in front of each proposed temporary wall. Each one will be installed to a depth of 80 feet.
	Settlement Transducers (12 each)	Installed at the base of the granular material 25 ft. and 125 ft. back from abutment wall face, placed at centerline of roadway and one each at shoulder/ driving lane.
	Pore Pressure Transd. (8 each)	Installed 25 ft. and 125 ft. back from the abutment wall face, below centerline of roadway and placed in the middle of the two most critical fine grained layers.

Structure	Instrumentation Type	Instrumentation Location
Drainage Structure surcharged areas	Settlement Transducers (20 each)	Installed 3 feet into natural ground below centerline of roadway and centerline of pipe and one each at the shoulder/ driving lane and pipe centerline intersection.

The rate of temporary fill and surcharge construction was to be ultimately be determined by the pore pressure, settlement transducer, and slope inclinometer field readings. Time for the removal of the temporary fill and surcharge was ultimately be based on both the amount of pore pressure dissipation and the rate of settlement of the foundation material.

Properly trained Highway Department personnel took and recorded instrument readings at the following recommended intervals:

Transducer readings - At the beginning and end of each day of fill construction in an instrumented area. Once a week thereafter until drainage structures are installed and the temporary walls are removed.

Slope inclinometer readings - At the end of each day during construction of the temporary walls and surcharge. Once a week thereafter until the final abutment wall construction had been completed.

Construction Considerations

1. A temporary wall and surcharge was constructed at each abutment in order to consolidate the foundation soils beneath the embankment. The temporary wall was constructed with a geogrid system to be able to withstand the differential settlements.

2. A geotextile fabric wrapped granular material was needed to be constructed between the natural ground and the roadway embankment. We recommended that the bottom geotextile be a reinforcing fabric having minimum engineering properties such as Mirafi 500x. We recommended that the granular material have a free draining gradation, and a thickness of 8 inches.

3. We recommended that the abutment areas and drainage structures at Station 73+00 and 78+00 that require wick drains to accelerate consolidation settlement, have a change in wick spacing to provide a smooth transition from the treated areas to the untreated areas.

4. In the area of the overpass and the 40 foot embankment area, it was important that the rate of construction for the embankment be controlled in such a manner that excess pore pressures are not generated that would produce a shear failure. Figure 7 in the Slope Stability Section of this report was used as a guideline for such a rate.

5. We recommended that drainage structures (other than pipes at Station 73+00 and 78+00) should be surcharged with 10 additional feet of embankment. The full height of the surcharge extended 25 feet either size of the drainage structure.

Construction Sequence

1. Areas to receive granular material were cleared and grubbed.

2. Central Lab, Geotechnical Section, New Mexico State Highway & Transportation Department installed settlement, pore pressure and inclinometer instrumention. Areas to have wick drain installation had instrumentation installed immediately after wick placement.

   18" below bottom of leveling block elevation were excavated for permanent reinforced earth wall. 24" lengths of 18" CMP sleeves and caps were installed.

3. Geogrid (in overpass area), reinforcement fabric and drainage aggregate around 18" CMP sleeves were placed.

4. Reinforcement or separator fabric, depending on embankment height was placed.

5. Granular material was placed.

6. Wick drains in accordance with plans and Special Provisions (Overpass area) were installed and instrumentation was placed immediately after wick drain installation.

7. Separator fabric over drainage aggregate was placed.

8. Temporary embankment and surcharge according to embankment height schedule as shown in plans was constructed.

9. Trained highway personnel, under the direction of the Central Lab, Geotechnical Section, monitored settlement and pore pressure instrumentation.

10. After 90% consolidation settlement was achieved as determined by Geotechnical Section, additional foundation samples were obtained by New Mexico State Highway & Transportation Department for conformation of increased soil strengths at the abutment sites. Ninety percent of consolidation settlement in the wick drained temporary wall area was estimated to take place in 130 days.

11. Temporary wall was removed.

12. 120 inch culvert was installed.

13. Final reinforced earthwall was constructed.

Post Construction Observations

Recorded Settlements

The recorded settlements in the railroad overpass abutments are listed in Table 6. The recorded settlements were not quite as high as expected although the settlement curves had shown that the 90% consolidation target had been obtained.

Station	Settlement (in.) Lt.	Cl.	Rt.
73+00	6.0	10.5	14.0
73+64	7.5	6.0	6.0
74+45	8.0	12.0	9.0

Table 6

The great variability of the soil types and thicknesses are thought to have attributed to the lower settlement values.

Recorded Pore Pressures

The recorded pore water pressure increases were monitored by placement of pore water pressure transducers in CL material at three locations: Stations 73+00, 73+64, 75+00. Two transducers were placed at each of these location at depths of approximately 15 and 45 feet.

The increases in pore water pressure were measured to be about 5 PSI in most of the soil layers at their highest point. They then subsided to a point just above their original pressure after about 90 days except for two of the layers. These two layers showed pressures significantly lower than the original starting pressures. No explanation was found for this result. The starting pressures were close to the static water level pressures.

GEOGRID-EXPANSIVE CLAY EMBANKMENT

By

Sam I. Thornton and Michael S. McGuire

ABSTRACT

An 80 feet high by 800 feet long geogrid reinforced highway embankment was constructed in 1988 by the Arkansas Highway and Transportation Department. Side slopes of the embankment are two horizontal to one vertical (26.5 degrees). The embankment is located at Cannon Creek about 20 miles southeast of Fayetteville on state highway 16.

Cannon Creek embankment is the tallest in the USA built by a Department of Transportation. The embankment is also unusual because it is built with a clay fill.

The clay fill used in the embankment is expansive in nature being classified AASHTO A-7-6 (37). Liquid limits average 65 and the plastic index averages 40. The fill was placed in 8 inch lifts and compacted to 95 percent of standard compaction effort (AASHTO T-99).

No slide failures have occurred in the embankment. The maximum measured strain in the geogrid is 1.3 percent, well below the design maximum strain of 10 percent.

Contrasted with the stable geogrid reinforced embankment is a failed backslope cut at a five horizontal to one vertical (11.3 degree) slope. The failed backslope is in an undisturbed deposit of the same clay that was used in the embankment.

INTRODUCTION

The Arkansas Highway and Transportation Department (AHTD) has constructed an 80 feet high by 800 feet long geogrid reinforced embankment on state highway 16 at Cannon Creek (Figure 1). The embankment, located about 20 miles south east of Fayetteville, Arkansas, (Figure 2) is the tallest in the USA built by a Department of Transportation.

MATERIALS

Clay

The Cannon Creek embankment is particularly interesting because it is constructed with highly plastic clay {AASHTO SOIL CLASS: A-7-6(37)}. The Atterberg limits are high, the liquid limit is from 50 to 80, and the plastic index is from 30 to 50

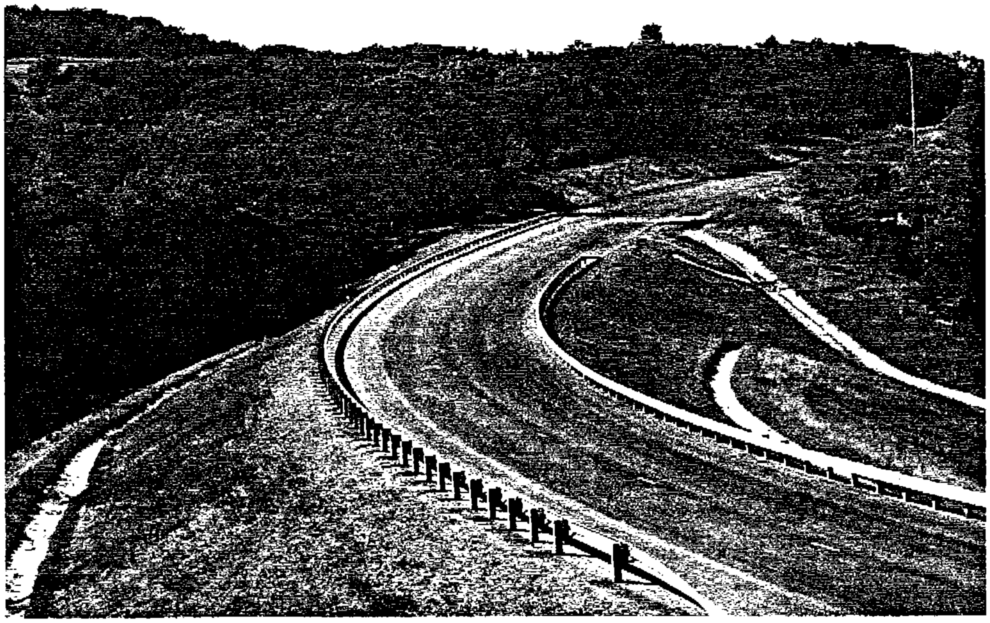

Figure 1

The Cannon Creek Embankment

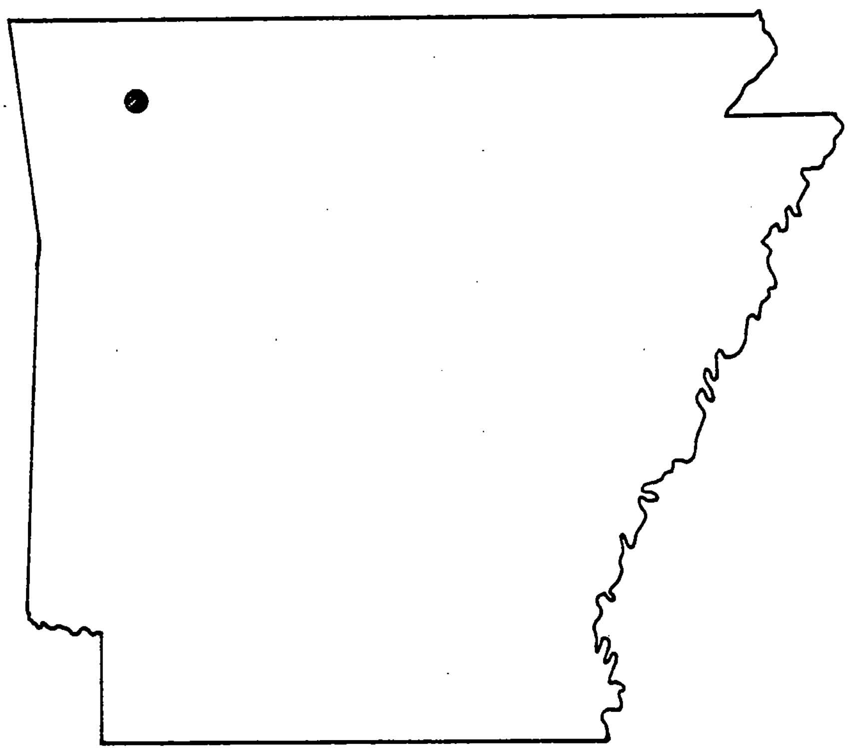

Figure 2

Cannon Creek Embankment Location

(AHTD,1988). The clay has an expansive nature causing the soil to increase in volume when it has access to water.

The soil was compacted in eight inch lifts, to 95 percent (AASHTO T-99) relative density (moist unit weight of 120 pcf). For stability, the internal friction was assumed to be 20 degrees and the cohesion 50 pounds per square foot (Table 1).

TABLE 1

Cannon Creek Soil Properties

Compacted Clay Fill			Foundation Material (bedrock)		
Φ	=	20 deg	Φ	=	45 deg
c	=	50 psf	c	=	1000 psf
Ym	=	120 pcf	Ym	=	120 pcf

Φ = angle of internal friction
c = cohesion
Ym = unit weight

Geogrid

The geogrid reinforcement was a high strength polyethylene manufactured by the Tensar Corporation of Morrow, Georgia. Both uniaxial reinforcement, designated UX, and biaxial, BX, reinforcement were used in the embankment. Strength of the reinforcement are contained in Table 2.

TABLE 2

Geogrid Reinforcement Properties

TYPE	Tensile Strength
UX1400 (SR1)	1,200 lb/ft
UX1500 (SR2)	2,200 lb/ft
UX1600 (SR3)	3,000 lb/ft
BX1100 (SS1)	750 lb/ft

Uniaxial geogrid was used as primary reinforcement for internal stability (Figure 3). The biaxial geogrid was placed near the surface to give stability to the embankment face.

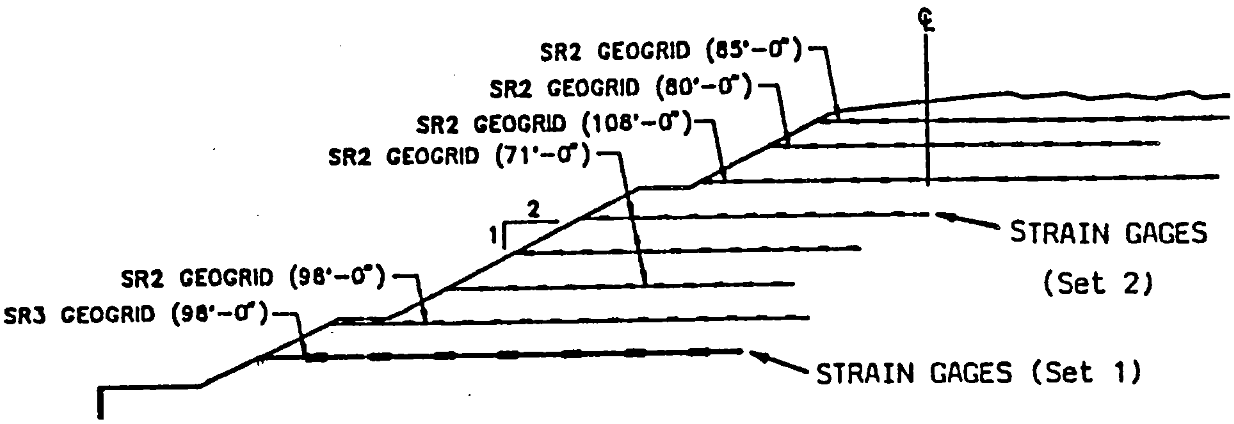

Figure 3

Embankment Reinforcement

STABILITY

The embankment was designed, based on assumed shear strength, by Tensar for a minimum factor of safety of 1.30. Tenslol, a computer program, was used in the design (Tensar,1985). A typical critical failure surface is shown in Figure 4.

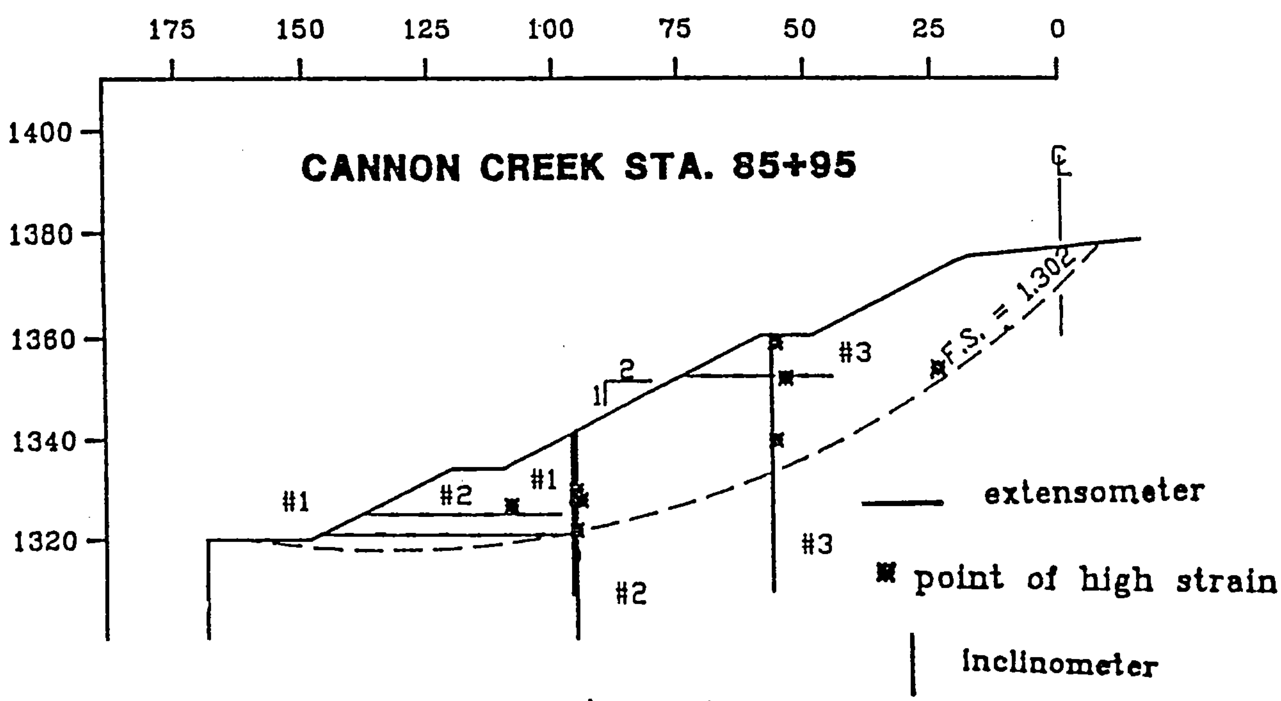

Figure 4

Failure Surface

In order to check the design, the embankment was instrumented with sixty-seven (67) strain gages, three (3) extensometers, and three (3) slope inclinometers (Figures 3 and 4). Points of high strain are shown in Figure 4. The maximum measured strain in the geogrid is 1.3 percent which is well below the maximum design strain of ten (10) percent. Details of the strains which were measured in the first year after construction can be found in the thesis by McGuire (1990).

Side slopes of the embankment face are two horizontal to one vertical (26.5 degrees) (Figure 4). To date, Summer 1990, no failures have occurred in the embankment. Figure 5 is a photograph with the Cannon Creek reinforced embankment in the foreground. The upper right portion of Figure 5 is of a failed backslope cut at five horizontal to one vertical (11.3 degrees) into the same clay that was used in the embankment. Figure 6 is a closer view of the failed section.

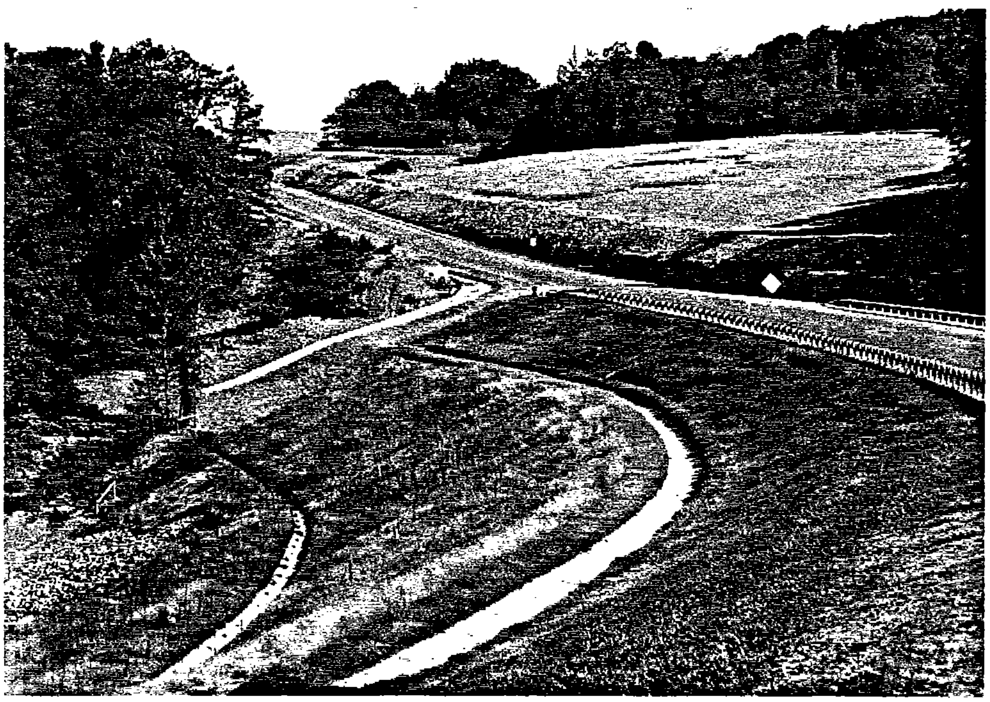

Figure 5

Embankment and Failed Backslope

Figure 6

Failed Backslope

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CREATING AN ELEVATED CATCHMENT AREA USING A PRECAST MODULAR WALL SYSTEM

Richard H. Cross
Associate Engineering Geologist
New York State Thruway Authority

ABSTRACT

Conditions sometimes exist that make stabilization of a rock slope so expensive or so disruptive of traffic that it becomes more economical to employ extensive protective measures. Four rock slopes along the New York State Thruway within the Palisades Interstate Park presented just this type of situation. The high traffic volumes experienced in this area made remedial efforts imperative, however, they also made any major remediation nearly impossible. Tight Right of Way within the park area further complicated the problem.

After a comprehensive study of both the rock slopes and the site conditions it was decided to construct a wall of sufficient height to create an adequate catchment area on the backfill at the top of the wall. Site conditions dictated that the structure meet several design and construction requirements.

A. The structure must have a narrow footprint in order to fit in the available space and still retain a full width shoulder.

B. The structure must provide a high degree of security against slide debris reaching the pavement.

C. The structure must combine a high level of structural stability with an aesthetically pleasing appearance and low maintenance.

D. The system must be able to be constructed from a limited staging area.

A wall system based on tied back soldier piles was selected as best meeting these requirements. Wall units consisting of interlocking precast concrete modules that slide over the piles are used instead of conventional lagging. Grouting of the annular space around the piles protects the piles from the salty environment as well as transferring the load from the wall units to the piles. A wall top fence system using a wire rope/net catchment fence was incorporated to both reduce the required height and stop splatter from rock falls. These walls are currently under construction with completion scheduled for late spring 1990.

INTRODUCTION

The New York State Thruway is a 559 mile toll road that makes up the backbone of the Interstate highway system in the state. Running north from New York City to Albany and then west to Buffalo and the Pennsylvania state line the highway is sometimes referred to as the "Main Street of New York". Following a rockslide related fatality in

January 1988, a program was established to evaluate all rock slopes along the system. As a result of these evaluations thirty sites were identified as warranting immediate remedial work. Recommendations for remediation of the slopes made during this initial evaluation process targeted temporary measures that could be used to secure the slopes pending permanent remediation. The temporary measures for all sites consisted of scaling and covering the slopes with draped wire mesh. Following the receipt of these recommendations a site by site assessment was made to determine the cost effectiveness of the short term remedial approach. Also considered at this time were constructability problems and maintenance and protection of traffic issues. The four rock slopes that comprise this project were set apart as having multiple problems in these areas.

SITE CONDITIONS

The project area lies in Rockland County, thirty to forty miles north of New York City, on a portion of the highway that serves as the main commuter road for the area. The two way AADT for this segment of the highway is approximately 59,000 vehicles with a high seasonal variation in the summer months. The highway lies at the foot of rock ridges that form the eastern flank of Ramapo River valley. The ridges are composed of highly contorted gneissic rock with major north-south trending fold axes. The combination of this structure and extensive weathering has produced the valley that the river and highway follow. Because the highway runs generally parallel to the ridges most of the cuts along this section are sidehill cuts on the east side of the northbound roadway. The highway in this area is a six lane section and was constructed with a grade separation of between five and fifteen feet between the northbound and southbound roadways. Extending through the entire ten mile length of the project area the grade separation presents a major obstacle to detouring traffic around work zones. Constructed between 1953 and 1955, prior to the emergence of presplitting as a widespread blasting technique, the rock slopes were built to standards that were state of the art at the time. The slopes were built with a nominal inclination of three vertical on one horizontal however several of the sites have extensive areas where the actual inclination is vertical or overhanging due to the rock structure. All of the slopes show signs of considerable overblast. This damage frequently extends twenty-five to thirty feet back from the face. Slope heights range up to one hundred feet high with the back slopes extending far beyond the right of way limits.

INVESTIGATION

Quantity estimates were undertaken to determine a cost comparison of scaling versus recutting. Determining an accurate estimation of scaling quantities necessary for the cost effectiveness determination was extremely difficult due to the highly jointed and weathered nature of rock faces and steep backslope conditions. In some areas, marginally stable talus slopes at the top of slope presented obvious problems. In most areas, however, the backslopes were covered with overburden or vegetation making a visual assessment of backslope conditions impossible.

Due to right of way problems access to these areas for exploration could only have been obtained by pioneering a road through adjacent park lands. Time constraints made obtaining right of entry onto the park lands something to be avoided if at all possible. The best estimates that could be made based on observable conditions placed the volume of material to be scaled high enough that the cost for scaling the slopes would be very close to that of recutting the slopes. If some worst case scenarios proved true scaling costs could exceed those for recutting and still leave unstable conditions.

Constructability also presented severe problems. The large amount of rock to be scaled as well as the height of the slopes would require access to the top of slope areas by heavy equipment. This again could only be obtained across park lands. Even with top access the mid regions of the high slopes would have been only marginally accessible. Disposal of the excavated material also presented a serious problem.

As significant as these issues were they could have been resolved relatively simply by acquiring right of entry or additional right of way. Efforts along these lines would probably have been ultimately successful although potentially delaying the project for several years. The issue that posed the greatest problem and ultimately drove the design of the project was the maintenance and protection of traffic during the progression of work. The height, steepness and narrow setback of the slopes dictated that the northbound roadway be closed to progress the work. These conditions and the previously noted grade separation meant that a detour would be required and that it would need to extend the entire length of the project, approximately ten miles. Cost for construction of the detour was estimated at approximately seven million dollars. Width restrictions posed by several bridges along the southbound roadway limited any such detour to four eleven foot lanes. Traffic volumes through this area however, mandated that six lanes of traffic be maintained especially during the summer months when the project would be running. Since they could not be implemented rapidly enough both sealing and recutting were eliminated as possible remediation schemes.

Due to continued pressure for immediate remedial action, consideration was given to constructing of a wall of sufficient height to create an adequate catchment area on the backfill behind the wall. Early site investigations had indicated that the lower slopes at all the locations were relatively sound and stable. Encouraged by these indications a more detailed study of the sites was undertaken. In addition to the normal studies of foliation planes and joint sets done for the preliminary investigations photogrametric cross sections had been taken. These were now used to assist in determining how high a wall would be required to obtain a catchment area of adequate width at each of the areas. Maintenance records were also used to provide information of previous rockslides such as size and where the slide had impacted. The result of this study indicated that the wall concept was workable for these locations. While there were some indications that slides of sufficient volume to fill or exceed the capacity of the catchment area were possible, it appeared that all of the slides to date could have been retained using this system. It also appeared that catchment fence of sufficient strength could be constructed at the top of the wall to retain slides of the magnitude indicated possible by the study. It should be noted that the rockslides experienced at these locations tended to have maximum volume of three cubic yards with a maximum particle size of two cubic feet. The maximum projected slide volume was approximately ten cubic yards.

DESIGN

Following the determination that the wall scheme was feasible for these slopes, four criteria were established for a wall design.

A. The structure must have a narrow footprint in order to fit in the space available and still retain a full width shoulder. Generally, the slope setback in the tight areas varied between seventeen and twenty feet from the traffic lanes. There was, however, a short section where the setback was as little as fifteen feet. Although some minor excavation was possible, in general, the wall had to fit in this space and still provide room for a ten foot wide shoulder.

B. The structure must provide a high degree of security against slide debris reaching the pavement. Although most of the slides experienced were under one cubic yard in size the energy levels were high enough to enable slide debris to reach the southbound lanes.

C. The structure must combine a high level of structural stability with an aesthetically pleasing appearance and low maintenance. Since the walls were being constructed in a park area it was felt that an effort should be made to make it blend in with the environment as much as possible. Also of great concern was the effect that years of exposure to salt and other de-icing chemicals would have on the exposed surface and structural members of the wall system.

D. The system must be able to be constructed from a limited staging area. Due to the grade separation between the two roadways the maximum lane shift that could be accommodated without major effort was ten feet. This provided a minimum work zone width of twenty three feet behind the concrete barrier.

A wall system based on tied back soldier piles was selected as best meeting these requirements. The 12x84 steel piles are set on twelve foot centers in twenty four inch diameter predrilled holes. The piles are set with a twenty four vertical on one horizontal batter. The pileholes are required to be drilled six feet into rock to offset the sidehill nature of the cuts and the effects of overdrilling from the original highway construction. The piles are designed with splices for ease of installation and to provide tieback points. Tiebacks are one and three eighths inch epoxy coated Dwidag bars. The grade 150 bars are coupled approximately one foot out from the rock face. Extensions are installed between the rock face and the piles and covered with plastic pipe sleeves to protect them during the backfilling process. Vertical spacing of the tiebacks is either eight or twelve feet. One and one half inch crushed stone is used as backfill material up to a level five feet below the top of the wall. The last five feet of backfill is select granular material placed over a geotextile membrane. A stone lined interceptor ditch is constructed within this layer to prevent surface runoff from entering the backfill material. Some DIs were raised to the top of wall elevation to further enhance rapid drainage in high flow areas and also to prevent long flat runs of the ditch. Underdrain pipes are installed in the backfill material at the base of the wall and also below the top of slope ditch. The wall modules are twelve by twenty four foot precast concrete units. Each unit is cast with two twenty one inch round pile holes. The units vary in thickness from seventeen inches at the center and ends to thirty two inches at the pile holes. Non-structural interlocks are cast in the units to aid alignment during erection. Half height modules are used to offset ground elevation changes and tapered modules are used at the ends of each course. Heavily reinforced, each full unit weighs approximately twelve tons. All units have an exposed aggregate architectural finish on the face and are cast using a micro-silica admixture (5% by weight). Following placement of the units the annular space around the pile is filled with concrete to provide load transfer to the piles. This encapsulation also serves as corrosion protection for the pile. Wall heights range up to thirty six feet.

A cable net catchment fence was designed for installation at the top of the wall. The fence panels have an eight inch square mesh size and are woven with one quarter inch diameter wire rope with a five sixteenths inch diameter border rope. The mesh ropes have pressed on non moveable clips where they cross. The fence height is either twelve or eighteen feet. The panels are hung on 8 x 8 x 48 wide flange posts on twenty foot centers. The posts are tied back at both the top and at the post anchors. The mesh panels and posts are further restrained by five eights inch diameter supporting ropes at the top and bottom of the panels. Breaking elements installed on the supporting ropes and on the upper post retaining rope. The fence is set four feet back from the face of the wall to allow room for the fence to react under load without losing debris at the bottom. The fence system was designed to act independently from the wall.

CONSTRUCTION

In late September 1989 a contract was let for construction of the walls. The 13.8 million dollar contract called for the erection of approximately one hundred ten thousand square feet of wall at four sites to be completed by mid May 1990. Still not complete the construction of the walls has gone relatively smoothly though slowly. The most significant problems have been related to the layout and drilling of the pileholes. Inaccurate layout and drilling related difficulties resulted in twenty one of the first seventy six pileholes having to be plugged and redrilled. The contractor excavated and set an oversize sleeve to advance his pileholes through the overburden. His drilling subcontractor then occupied these locations assuming their accuracy. The misalignment problem that resulted from this procedure was two fold in origin. First, the location and/or verticality of the sleeves was not accurately set. Second, initial efforts to collar the holes and hold the correct location while drilling through fragmented rock at the overburden rock interface proved inadequate. Once the holes were being accurately located and the sleeves were being set with a concrete starting pad the misalignment problem virtually disappeared. The effect of this otherwise relatively minor problem was significantly enlarged by the contractors having set seventy six sleeves prior to starting to drill the pileholes. No serious problems related to the setting of the wall units has been experienced.

SUMMARY

In addressing the stability problems at these sites the Thruway Authority was faced with a dilemma. The same traffic volumes that made remedial work mandatory also placed severe restrictions on the techniques that could be used. With the cost for stabilization approximately equal to that for these protective measures, the ability to maintain traffic became the deciding factor. As part of the design process some judgements have been made as to the potential size and frequency of rock falls that could be expected. These walls have been designed to provide a level of protection adequate to restrain rock falls up to the largest foreseeable size based on these judgements and not sustain critical damage. We believe that this design presents a suitable state of the art approach to the difficult problems experienced at these sites. Time, as with all remedial efforts, will ultimately be the judge of the appropriateness of this design.

17 MILES TO MOUNT ST. HELENS: OPERATIONAL ASPECTS OF THE GEOTECHNICAL INVESTIGATION

David P. Findley, Senior Engineering Geologist, Golder Associates Inc.
George B. Deardorff, Principal, Golder Associates Inc.

ABSTRACT

The May 18, 1980, eruption of Mount St. Helens destroyed a major portion of Washington State Route (SR) 504 located in the valley of the North Fork Toutle River. Golder Associates Inc. was retained by the Washington State Department of Transportation (WSDOT) to conduct a geotechnical engineering investigation of a 17 mile portion of the new Spirit Lake Memorial Highway, including seven bridge locations, that will replace the destroyed highway. This paper presents the operational methodologies, challenges, and lessons learned during the geotechnical investigation program.

The program was completed in 108 days of continuous operation during the winter and spring months of 1987. The inherent wet and snowy weather conditions of the Mount St. Helens area; the safety consideration associated with the proximity of the volcano; and the destroyed timber covering the mountainous terrain along the centerline of the proposed alignment all required careful coordination of personnel and equipment to meet the objectives of the program. The five drill rigs used in the investigation program completed 382 boreholes totaling 17,194 feet of soil and rock core samples. In addition 250 standpipe piezometers, 205 test pits, and 18,704 feet of seismic refraction line were completed to complement the investigation program.

The specific methodologies associated with the program are presented including a discussion of the development of the drilling program, bid specifications, and selection of the drilling contractor. Comments are made on the importance of a remote base camp as a cornerstone of the program and how telecommunications aided the efficiency of operation between the field forces, base camp, and the engineering home office. The importance of site safety programs are highlighted. Summaries are presented to document drilling penetration rates for the various materials encountered and the numbers and types of personnel and equipment required to complete the project. Comments are also made concerning the drilling of remote boreholes requiring the assistance of helicopter support, the drilling of angle boreholes, and new techniques to produce oriented rock core samples.

An important aspect of the operation was the establishment of on-site engineering facilities. Details are presented describing the physical and logistical challenges in establishing such an operation in a remote location including scheduling of personnel and daily work flow, and contract administration. A summary is presented describing the lessons learned during the program.

INTRODUCTION

This year marks the tenth anniversary of the eruption of Mount St. Helens. The eruption which occurred on May 18, 1980 was a natural event of unprecedented proportions in modern history. On that quiet Sunday morning the mountain claimed the lives of 60 people, destroyed 200 square miles of productive timber land. It also buried the Toutle River valley and destroyed Washington State Route 504 (SR 504) beneath a volcanic mud flow resulting from the eruption. The former route of SR504 originates in Castle Rock Washington along Interstate 5, heading east and terminating at Spirit Lake located on the north side of Mount St. Helens.

The Washington State Department of Transportation (WSDOT), in conjunction with the Federal Highway Administration (FHWA), began preliminary design studies for a new highway, to be re-named the Spirit Lake Memorial Highway, in 1985. The general scheme of the re-alignment was to move the highway from the floor of the valley to the north slopes of the Toutle River valley. A preliminary route was selected in 1986. The new SR 504 is to consist of 25 miles of two lane paved roadway and eight bridges, see Figure 1.

By a competitive bid process, Golder Associates Inc.'s Redmond Washington office was awarded the contract by WSDOT to conduct the geotechnical engineering investigation along the final 17 miles of the proposed alignment. This highway segment would begin at around an elevation of 1500 feet, rise to an elevation of 3800 feet at Elk Rock and descend to an elevation of 3100 feet at a turnaround at the end of the highway at Coldwater Lake.

The proposed geotechnical field investigation program developed for the project consisted of 225 alignment borings, spaced roughly 300 to 400 feet on centers. In addition 155 borings were planned for proposed bridge piers, abutments, retaining walls, and borrow source evaluations. At least 40 of the planned borings were expected to require helicopter support because of difficult access. Two phases of drilling were planned. The initial phase, termed primary borings, was drilled on the first pass over the alignment. Secondary borings were drilled on a second pass and were used to fill data gaps along the alignment once all the primary borings had been drilled and data insufficiencies had been identified.

A major requirement of Golder Associates contract with the WSDOT was a minimum Minority Owned Business Enterprise (MBE) and Woman Owned Business Enterprise (WBE) participation in the project. The minimum requirements were 10.1% MBE and 6.0% WBE participation. To satisfy these requirements, Golder Associates subcontracted portions of the work to appropriate firms.

Drilling Program Development

The drilling program was developed only after an extensive field reconnaissance and mapping program was completed by Golder Associates to indicate geologic, and topographic conditions that would be encountered during the actual field investigation program. In addition, the highway alignment was reviewed by an independent drilling consultant and senior Golder Associates personnel to identify potential problem areas. Consideration was

Figure 1

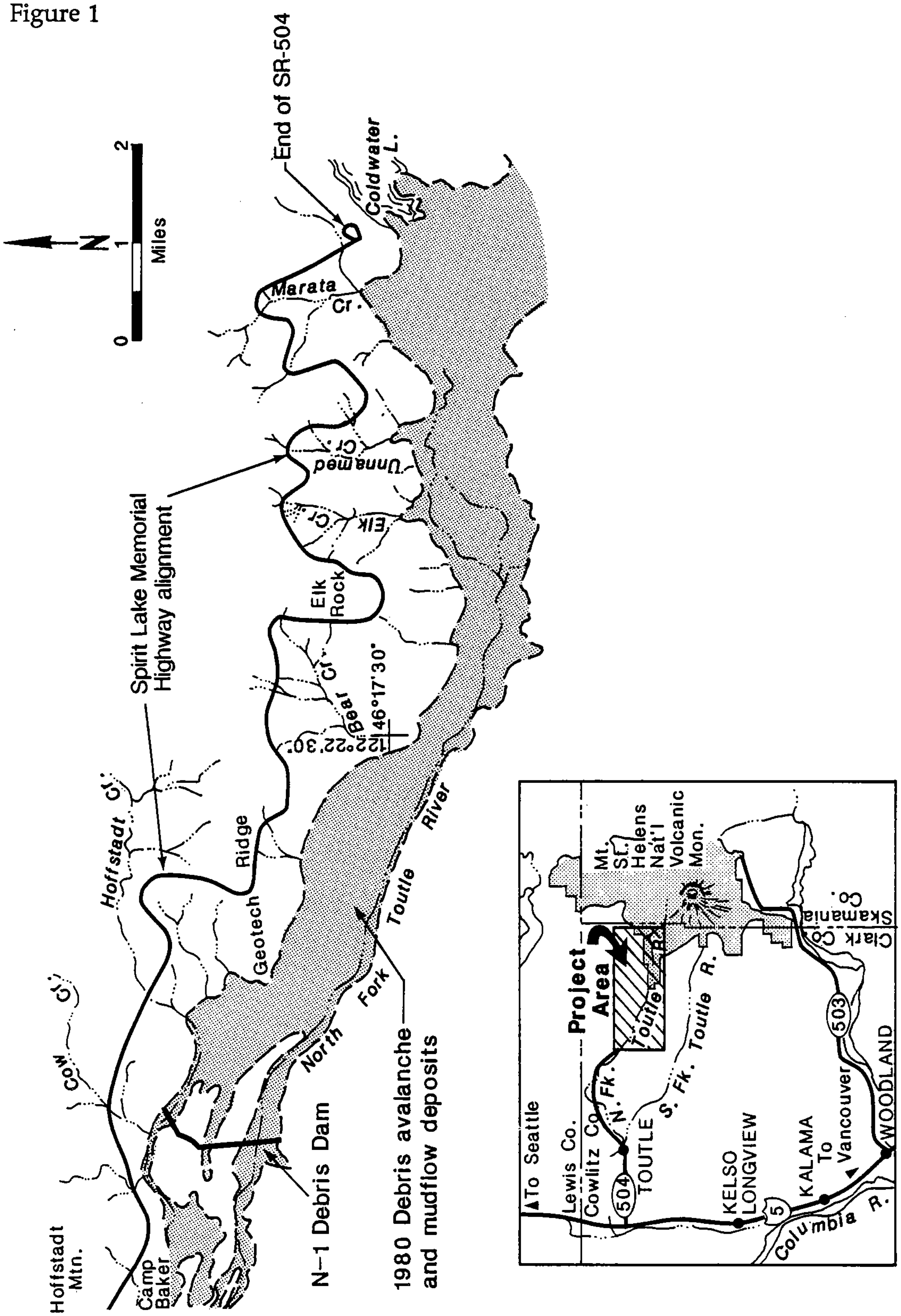

Figure 1
Location Map

given to how surface volcanic ash deposits, the existing network of logging roads, and overturned timber (blow down) would affect the program.

A Golder Associates' prepared bid package, which had been reviewed and approved by WSDOT, was supplied to pre-qualified drilling contractors after submitting a $100 bid package fee. Bidders on the drilling contract were required to attend an on-site prebid meeting and submit a technical proposal on how they would accomplish the job. In addition, prospective bidders were required to post a performance bond as well as supplying detailed resumes of proposed personnel.

To evaluate the bidders proposals, Golder Associates developed an Engineers Estimate. The Engineers Estimate was based on a careful analysis of previous field work conducted for a preliminary routing study from which drilling rates in various geologic conditions were derived. From this evaluation, Golder Associates detailed field reconnaissance and the combined corporate database of experience, the number of drills and personnel, and logistical requirements to complete the field portion of the project in the required 120 day time frame was estimated.

One of the major elements in developing the field program was the logistical support required to house and feed the project field team for the project. Primary factors considered were that the project would be conducted during the winter months in a remote mountainous area and the nearest community which could support the anticipated man-power requirements was Castle Rock, located approximately 20 miles from the beginning of the project at the junction of Interstate 5 and State Route 504, see Figure 1. The decision was made to conduct the field investigation from a remote project base camp based on consideration of the following factors:

- Inherent unsafe conditions with project personnel travelling from Castle Rock to the project site twice daily (50 mile minimum round trip) during the winter months along snow-covered winding mountain roads.

- Cost effectiveness related to reduced on-site effectiveness due to long daily commutes vs. a base camp with minimum travel time and increased on-site work time.

- The on-site engineering capability allowed the elimination of a planned intermediate mobilization between the drilling of the primary and secondary borings, resulting in a substantial cost savings to the project.

Miscellaneous drilling Contractor Bid Specification Notes

To meet the required completion time of 120 days for the field investigation, the drilling contract contained a liquidated damages clause stating that the drilling contractor was liable for the daily fixed operating costs ($2,000/day) of maintaining the base camp for each day past the 120 day deadline that the work was delinquent. In the end, the project was completed in 108 days and no liquidated damages were assessed.

In order to accommodate conditions beyond the Engineer's control, Golder Associates reserved the right to declare Non-working days. This clause was intended primarily to cover weather conditions such as severe snow which would prevent performance of the field investigation. These non-working days would not be counted as part of the 120 day contract performance time.

Drilling Contractor Selection

Five drilling contractors submitted bids for the work. The bids ranged from a high of $2,409,002 to a low of $830,208 with an average of $1,260,860. These values compare to an Engineer's Estimate of $854,000. Table 1 is a copy of the bid line items.

Golder Associates was under no obligation to award the contract based solely on the lowest bid. However, the lowest bidding contractor also submitted a superior technical proposal indicating track mounted drills along with a complete listing of equipment, personnel, supervision, and resources to do the job. In addition, their reference verification was excellent. A site visit to the drilling contractors shop and operations was made to verify the type and condition of the drilling equipment proposed for the project. As a result of the technical proposal and the lowest bid price, P C Exploration of Woods Cross, Utah was awarded the contract.

Golder Associates retained the services of eight firms to supply other essential project requirements. Exploration Products, located in Spokane, Washington, provided the turn key base camp operations; Romine Excavating, of Centralia, Washington, provided all drill access and pad construction, as well as conducting rippability and fill compaction tests; Halme Concrete Sawing, of Vancouver, Washington, supplied a courier shuttle service between the base camp and Golder Associates' Redmond, Washington office, an excavator for test pits, and a grader for snow removal and maintaining logging roads; Hong Consulting Engineers, Inc., of Lynnwood, Washington, provided additional geotechnical testing capability when the need arose; Techstaff, Inc., of Seattle, Washington, supplied on-site secretarial support; Geo/Recon International, Ltd., of Seattle, Washington, provided seismic refraction survey services; Geo/Resource Consultants, Inc., located in Seattle, Washington, supplied an additional geologist for drillhole logging and inspection as required by project demands, and finally Horton Dennis & Associates Inc., of Kirkland, Washington, provided on-call surveying services.

Drilling Program

Drill Site Access

Golder Associates provided all drill access and drill pad preparation through an MBE subcontractor, Romine Excavating of Centralia, Washington. Access to the majority of the drill sites was provided by bulldozers and track mounted excavators building short spur roads from the existing network of logging roads. Borings were located by stationing and offset by WSDOT and subsequently shot in surfaces elevations. With a few exceptions, borings were located spot-on. The usual equipment combination to build access roads and

TABLE 1

DRILLING CONTRACT BID ITEMS

Item	Description	Unit	Estimated Quantity
1	Initial Mobilization and Final Demobilization	L.S.	1
2	Intermediate Demobilization and Remobilization	L.S.	1
	Surface Rig Mobes		
3	- Primary Boreholes	each	369
4	- Secondary Boreholes	each	84
5	Helicopter Rig Moves	each	40
	Soil Drilling and Sampling		
6	- Primary Boreholes	per foot	4,000
7	- Secondary Boreholes	per foot	1,000
	Rock Coring and Sampling		
8	- Primary Boreholes	per foot	10,500
9	- Secondary Boreholes	per foot	2,500
10	Observation wells	per foot	9,000
11	Oriented Core	per foot	500
12	Drill Rig Standby	rig hour	200

drill sites consisted of a D-8 dozer, D-5 dozer, and a track mounted excavator. The excavator was equipped with a thumb which was hydraulically actuated and operated in opposition to the bucket allowing the excavator to pick up trees, boulders, etc. The D-8 was particularly useful in blow down areas with rugged topography.

The term "blow down" refers to areas of standing timber which were blown down like tooth picks in an imbricate pattern by the blast of the May 18, 1980 eruption. Depending upon the proximity to Mount St. Helens and local topographic effects, standing timber was either sheared above the ground surface leaving the stump in-place or the entire tree was blown over, uprooting the stump. Tree stumps were typically on the order of 3 to 4 feet in diameter.

The D-8 would pioneer the access road, dozing large stumps and provide rough grading, with the D-5 following behind completing the finish grading and drill pad preparation. The major problem with the above referenced equipment was soft ground conditions resulting from springs and seeps hidden by the blow down debris. Fortunately, both dozers were equipped with winches and, working in tandem, could free themselves when such conditions were encountered. The thumb-equipped track-mounted backhoe was particularly useful in areas of soft, wet ground; where it could easily construct corduroy road and place culverts across small streams to speed access road and drill pad construction. The most difficult access was in areas of steep topography in the blow down areas. The easiest was in areas of moderate topography and those areas which were logged prior to the eruption. In blow down areas, the rate of pad preparation ranged between 5 to 7 pads per day, while in the easier areas, 15 to 20 pads per day could be constructed.

An important consideration in access road and drill pad preparation was how far ahead of the drills the equipment would operate. Initially 13 pads per drill, spaced approximately 300 feet apart, were prepared. Many of the access roads deteriorated (became soft and muddy), as a result of inclement weather consisting of heavy rain and light snow. This deterioration would require additional dozer work to improve and maintain the prepared access roads or the dozers would be required to provide assistance in moving the drills. A D-3 LGP (Low Ground Pressure) dozer supplied by one of the WBE subcontractors, was particularly useful in maintaining access roads and trouble shooting problem areas. Based on this experience the number of pads prepared in advance of each drill was reduced to about five.

An added benefit was the dozer operator's local experience and ability to provide access to many of the drill sites which had been considered helicopter holes and inaccessible in the preliminary evaluation of the drilling program. The cat skinner (dozer operator) was able to reduce the number of original estimated helicopter sites by approximately one-half. In hindsight, it would have been prudent to have an experienced cat skinner review the drill sites where helicopter support was anticipated prior to establishing the drilling program.

One of the more substantial logistical challenges during the course of the field investigation was keeping the Toutle Valley road open. This road served as the major arterial dirt road from which the alignment was accessed from several secondary spur roads leading to the north slopes of the valley. The road crossed the Toutle River at a single location via a series of six 6 foot diameter culverts. A major rain storm hit the Mount St. Helens area on the eve of the first full day of the field program washing out the culverts and road. Without the Toutle Valley road, an additional 0.75 to 1 hour of travel time was added to gain access to the alignment.

The solution to the problem was the installation of a temporary bridge constructed of a railroad flat car to serve as a road bed and two large imported tree stumps acting as abutments. The two abutments were armored with rip-rap to retard erosion and undermining. The railroad flat car was attached with a wire cable to a "dead man" on one side to prevent loss of the flat car should the bridge wash out. Indeed, high flows in the Toutle River washed out the bridge on a subsequent occasion necessitating re-installation of the bridge. The time required to re-install the bridge using a D-8 and excavator was approximately three days.

Drilling Equipment

All drilling, except the helicopter supported holes, were drilled with track mounted drills. The mix of track mounted drills consisted of the following: two mobile B-40's, two mobile B-53's, and an ARDCO Model H.

All of the drills, with the exception of the ARDCO were mounted on caterpillar type steel tracked chassis, see Figure 2. The ARDCO was mounted on a special flat rubber tracked chassis. The ARDCO drill was unsuited for the rough and muddy field conditions encountered and was replaced by a Mobil B-53 mounted on a Bombadier, a cleated rubber tracked machine similar to a Nodwell and commonly used in the Alaskan and Canadian bush, see Figure 3.

The steel-tracked type mounted drills performed fairly well on firm ground. The highest daily production was obtained from these drills. All of these drills had fairly narrow tracks and thus high ground pressures resulting in frequent sinking or getting stuck in soft ground conditions. By far, the most agile machine was the Bombadier mounted drill. Its mobility and low ground loading were ideally suited for the conditions encountered.

Moving time between holes was highly variable, ranging from a low of about 15 minutes to as much as five to six hours. A project average was in the range of 30 to 45 minutes. Factors affecting moving time were topography, weather, ground conditions, and the drilling equipment.

To supply the drills with water, the contractor had an efficient system consisting of four 200 gallon trailer mounted storage tanks that were supplied two conventional rubber tired 4 wheel drive water trucks. The storage tank trailers were positioned on the nearest logging road and water line laid to the drill. An additional water supply system consisted of placing a gasoline powered pump in the nearest creek an laying 3/4 inch PVC pipe to the drill. To avoid frozen water lines during cold weather, pipes and pumps were drained at the end of each day.

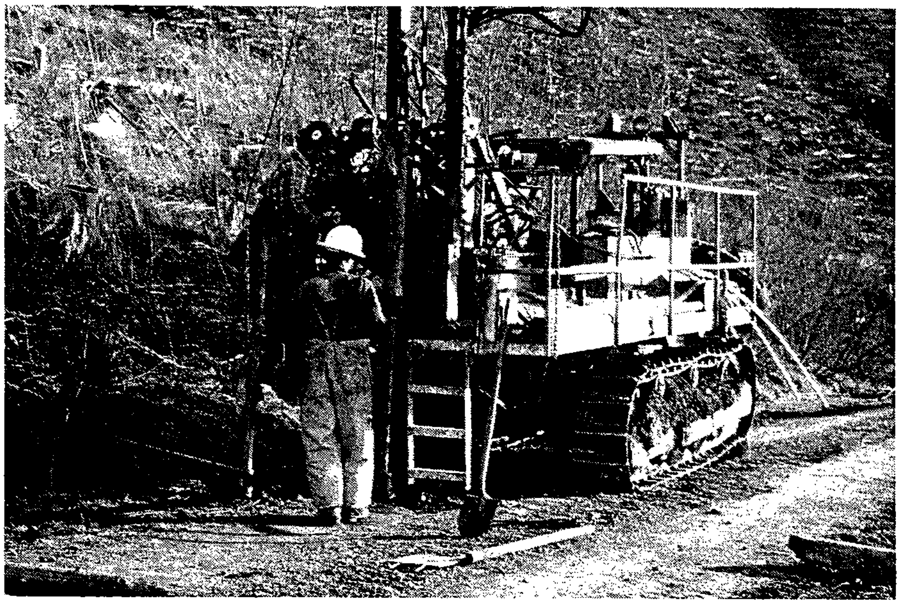

Figure 2. Steel Tracked Caterpillar Type Mounted Drill

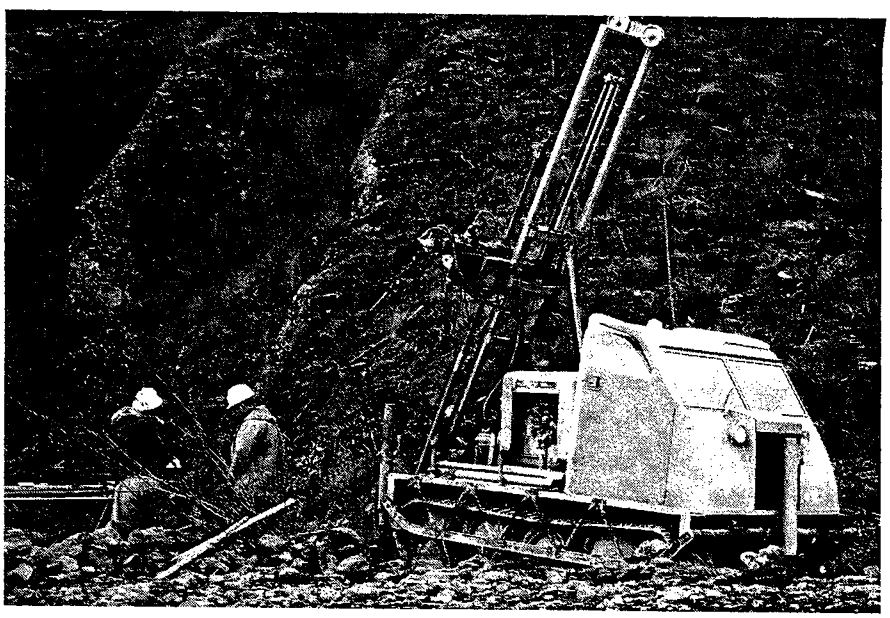

Figure 3. Bombardier Mounted Drill

A total of 14 helicopter fly holes were drilled. These fly holes were usually related to a planned bridge pier or abutment where extreme topographic relief precluded conventional dozer access. The drills utilized for these borings were skid mounted and consisted of the following: one Simco drill, one Joy 12, and one Sprague and Henwood drill.

These drills were transportable in one piece by a Bell 204 or Sirkorsky helicopter, see Figure 4. One fly hole set-up generally required three trips; one for the drill and two additional trips for drill rods, water tank, fuel drums, pumps, tools, and core boxes. Two helicopter fly holes were generally established together to reduce flying costs and required 1.5 to 2 hours of flying time. An important consideration in conducting safe and efficient helicopter operations for fly hole set-ups is the skill and experience of the pilot.

Drilling Methodologies

Several difficult drilling and sampling conditions were encountered on the project and the methods and techniques developed to maximize sample recovery are presented below.

All rock core drilling, except for a small amount of HQ size (2.5 in. O.D.) core, was N size (1.9 in. O.D.) wireline. The majority of the soils drilling was accomplished by advancing HQ size casing (3.5 in. O.D.) and driving split spoon samplers and shelby tubes, if appropriate.

One of the first problems encountered was poor core recovery in a hydrothermally altered tuff deposit. This unit is characterized by local zones of fairly competent rock in a softer matrix. Recovery from a conventional face discharge diamond bit was often poor, as the matrix was washed out by the action of the drilling fluid. The solution developed was to change to a single stepped, face discharge pilot bit designed by MINEX, a drilling supplies manufacturer, in the 15 stone/carat to 80 stone/carat range. In soft or broken zones, a 25 - 30 stones/carat bit proved most effective. This bit design effectively protected the core from the washing action of the drill fluid. Drilling with this bit design, combined with the use of a light polymer drilling mud and shorter runs, effectively solved the core recovery problem. The polymer mud mix consisted of MINEX 1330, Dris-Pac (similar to Quik trol) to increase mud viscosity, and soda ash to raise the pH so less polymer was required. In sound competent rock, an impregnated diamond bit with a soft matrix using plain water was used successfully to achieve acceptable core recovery.

A second major drilling and sampling problem was encountered in the alpine tills. These tills are coarser grained and contain more angular clasts than most of the tills found in the Puget Sound Lowland. As a result, hollow stem augers were ineffective in advancing the hole. The primary method of hole advancement was achieved by advancing HQ size casing with a Hastalite casing shoe to the specified sample depth and driving a split spoon sampler. Both standard size split spoons, driven by a 140 pound safety hammer, and large diameter California samplers, driven by a 320 pound safety hammer, were used. In the coarse grain tills, the larger diameter California samplers were generally more successful. Quite frequently a large boulder would have to be core drilled to obtain a driven sample of the surrounding soil matrix.

Figure 4. Helicopter Fly Hole Set-Up at Base of Hoffstadt Mountain

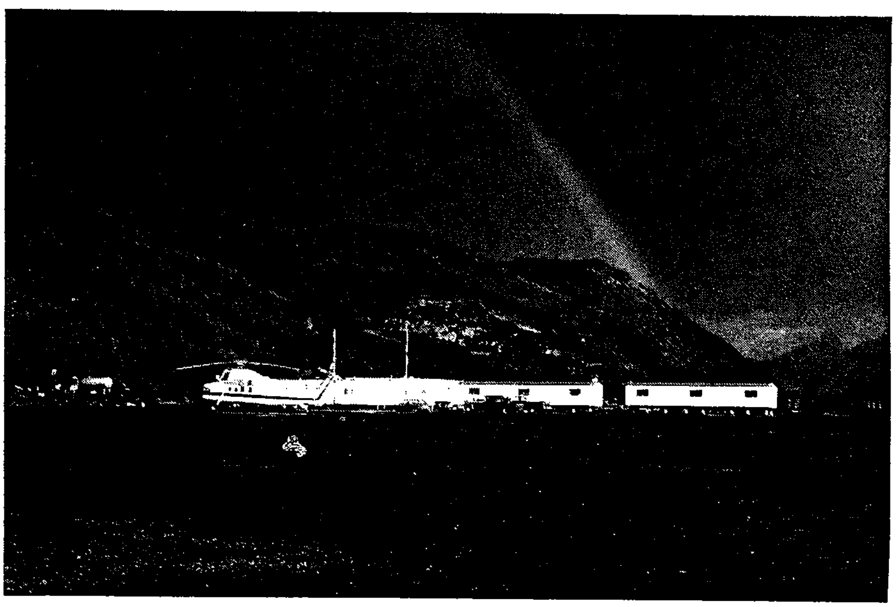

Figure 5. Spirit Lake Memorial Highway Project Base Camp at Camp Baker

Approximately 250 standpipe piezometers were installed in selected borings along the alignment. The piezometers were constructed of slotted 3/4 in. PVC pipe with the slotted tip placed in pea gravel. The monitored interval was then sealed with a bentonite pellet seal followed by grouting the remaining annular spare. These simple and inexpensive piezometers provided valuable information for stability and seepage analysis. A re-occurring problem during the project was getting the drilling contractor to complete the piezometer installations with an adequate concrete surface seal. In the future, the drill should not be allowed off the hole till the piezometer installation is complete. Inadequate surface seals required several passes along the alignment to complete piezometer installations.

Drilling Rates

Drilling rates were by nature highly variable. The main variables affecting drilling rates were lithology, RQD/fracture frequency, and drilling equipment. The primary geologic units drilled and sampled are summarized below.

Soils

- Loose to compact Alluvium consisting of a mix of fine to coarse sand and gravel
- Loose, poorly sorted, Talus and slide debris consisting of silt, sand, and angular gravel.
- Compact to dense, alpine glacial till poorly sorted sand and fine to coarse gravel in a silt/clay matrix.

Rock

- Hard competent andesite
- Relatively fresh unaltered tuff.
- Weak hydrothermally altered tuff containing local zones of relatively competent rock in a soft matrix of completely altered rock.
- An andesite unit that, upon exposure to the atmosphere, would slake and degrade. This characteristic was thought to have been caused by the expansion of the feldspars upon hydration.
- Andesite autobreccia

Soils which resulted in the slowest penetration rates were the alpine tills. A penetration rate of 5 to 7 feet per ten hour day were realized in some of the more difficult instances where large boulders within the till required coring to advance the hole. The slowest rock core drilling rates (in the range of 10-15 ft./ten hour day) were experienced in the hydrothermally altered tuffs. The highest single day production was 129 feet achieved in a fresh, competent, relatively unfractured andesite. Actual drilling rates were very close to those estimated in the field program development estimate.

Because of the soft, highly altered condition of some of the "rock" units and the extremely hard and dense nature of the alpine tills, a 50 blows/1 inch of sampler advancement criteria was applied to determine if soil drilling or rock coring rates would apply.

The project average for rock core drilling was 32 ft./day/drill and for soils drilling 13 ft./day/drill. These averages reflect the fact that most borings were completed in mixed geologic conditions. The average project boring depth was 45 feet and cost $43.80/ft. to drill. This cost includes footage rates, oriented coring, rig moves, helicopter rig moves, and piezometers.

Oriented Coring

Several of the planned higher cuts and bridge abutments along the alignment required quantitative structural geologic information such as; bedding dip and dip direction, for appropriate geotechnical design.

Quantitative subsurface structural geologic data was acquired by a custom built clay orienting core barrel. The orienting core barrel consists of a eccentrically weighted conventional core barrel containing a modified catcher design to hold and retain a plug of modeling clay. The eccentric weighting of the core barrel is achieved by lead placed within the lower half of the core barrel. An angle hole drilled at inclinations of 60 degrees or greater from the horizontal (around 65 degrees is optimum) is required to use the oriented core system. The oriented core barrel, with a clay plug in the catcher is inserted inside the rods and since it is eccentrically weighted, seeks the hole bottom. The outside of the core barrel is scribed with a line which defines the core bottom. Upon reaching the hole bottom, the clay makes an impression of the core stub from the previous core run. The core barrel is withdrawn, a conventional core barrel inserted, and a regular core run is drilled. Upon retrieval of the core, the core may be oriented from the clay impression of the core stub and scribe line. A goniometer is then used to record structural data.

The key factors for successful oriented core drilling are summarized below.

- The rock must be reasonably competent for the technique to work. Weak, friable rock is not conducive to obtaining good core stub impressions.

- The weighted core barrel must be able to rotate freely within the drill rods when lowered into position so the core barrel may seek hole bottom.

- The quality of the clay impressions is dependent upon the nature of the broken core stub from the previous core run. A high angle or irregular stub provides better impressions than broken core stubs oriented sub-parallel or sub-perpendicular to the core axis.

- Extreme care is necessary when piecing the core together to minimize erroneous data.
- Wherever possible, reference lines from successive core runs should be cross-verified by forward and backward extrapolation.

A total of 532 feet of oriented core was drilled along the project alignment.

Other Field Activities

While drilling and sampling along the alignment was the major field activity, several ancillary activities were on-going as well.

Extensive test pitting was completed and provided a cost effective method of expanding the geologic and geotechnical database. Cost savings were realized by substituting test pits for many of the planned secondary borings.

Geo/Recon, of Seattle, Washington, shot approximately 18,700 feet of seismic refraction line and some minor downhole seismic for top-of-rock approximations and velocity profiling.

Rippability and compaction tests were performed at several locations along the alignment with a D-7 dozer equipped with a ripper tooth. These data were made available to contractors for their estimating purposes.

In addition to the alignment drilling and sampling, two borrow source areas selected by the WSDOT were drilled and characterized. These borrow sources will supply sub-base material and select rock fill, and possibly concrete and asphalt aggregate.

Base Camp

As mentioned previously, the Spirit Lake Memorial Highway geotechnical investigation contained many logistical challenges. Chief among these challenges was the housing and feeding of the project team in a remote base camp located in mountainous terrain during winter months.

The project base camp was a very important element of the entire field program contributing to the successful and timely completion of the field investigation. The camp, located at Camp Baker, a former log yard, was in operation from January 31 through the end of May 1987. The base camp consisted of two major elements; life support services and technical support services.

The life support services were contracted to Myhre's Exploration Products of Spokane Washington. The camp was a "turn key" operation providing meals, sleeping accommodations, recreation, laundry, and shower facilities for the project staff (Golder Associates and all subcontractors). The camp was designed to house a maximum of 40 people per day with an average expected occupancy of 32 people. Accommodations for male and female staff was provided. The camp consisted of seven 12 x 60 foot ATCO skid mounted trailers assembled into two complexes, see Figure 5. One complex contained sleeping quarters, showers, kitchen and dining facilities, offices, and a recreation room. Recreational opportunities consisted of shuffle board game, video table game, card/pool table, satellite television and VCR, and a refreshment dispensing machine. The second complex consisted of sleeping, shower, and laundry facilities.

Exploration Products personnel included a camp manager and assistant, cook, and bull (assistant) cook. Electrical power to the camp was supplied by a trailer mounted diesel powered generator. Water supply was obtained from the Toutle River. Because of the high silt content, a three stage chlorinating filtration system was required to provide potable water. Sanitary requirements were met by four heated and lighted portable out houses which were serviced weekly. Gray water (laundry, kitchen, and shower water) were tight lined into a buried 2,000 gallon tank which was pumped and serviced as required. Meals consisted of "three squares a day." The meals were excellent and were a major factor in maintaining camp morale. "Conventional" meals were occasionally replaced by samples of international cuisine and outdoor barbecues to add some additional variety. The average cost per person per day for basic life support over the duration of the field program was $71.25 based on an average camp occupancy of 26 people. This daily per person cost compares to an estimated $160.00 per person per day if conventional life support services were utilized in Castle Rock.

The second element of the base camp consisted of the technical support facilities. These Golder Associates supplied facilities were comprised of two 12 x 60 foot trailers. One trailer served as a field laboratory and the second as office space. The field laboratory trailer was equipped to conduct, sieve analysis, Atterberg limits, slake durability tests, moisture contents, point load strength, jar slake tests, hydrometers, and rock core photography. The office trailer contained one desk top computer, two lap top computers, one laser printer, a desk top copier, and meeting space. The field office, located in the main camp complex, contained a second desk top computer, printer, desk top copier, and the communication center. The rental cost, including delivery and set up, of each trailer was $260/mo. Set-up of the trailers required inspection by the Cowlitz County Fire Marshall and Building Department before occupancy.

Golder Associates' vehicular needs for the projects were met by a fleet of leased 4 wheel drive trucks consisting of four S10 pickups for the rig geologists, two S10 Blazers (one for the field manager and a second for the project engineer's or geologists use) and a Suburban. A major operating cost of the vehicle fleet was tires and rims due to the severe road conditions which consisted of gravel logging roads. A set of radial truck tires was consumed in as little as 4,500 miles. During the course of the project, 14 tires and 5 rims were replaced. A mobile mechanic serviced the fleet on a monthly basis at an average approximate cost of $60/vehicle. The average cost of the seven vehicles including insurance was $6900/month.

Fuel needs for the project vehicles and camp generator were met by a local supplier who set up a fueling facility consisting of a series of above-ground gravity feeding tanks at no charge. The fueling facility contained three tanks to serve the needs of the field vehicles, camp, and drilling contractor. To satisfy regulatory requirements, a three foot high earthen berm was constructed around the perimeter of the tanks to contain possible leaks. The average monthly fuel costs for the field vehicles was $250/mo. at a delivered fuel cost of $0.919/gal.

Telecommunications

The project communication network consisted of two systems:

- An on-site two-way radio network
- A radiotelephone system for off-site communication

The on-site system consisted of a Motorola MCX 90 base radio with an effective radiated power (ERP) of 110 watts, eleven hand-held Motorola HT 90 (ERP of 5 watts) for field personnel, and two MCX 90 truck mounted radio (ERP 35 watts) mounted in two S 10 Blazers. The system operated on an assigned frequency of 152.93 MHz (VHF). In addition to these radios, the drilling subcontractor had a similar system operating on the same frequency. The performance of the on-site system was generally quite good. However, local topographic blind spots necessitated the relay of transmission when using the hand held radios. The truck mounted radios had a demonstrated range of approximately 20 miles.

The radio telephone system consisted of two units; a Motorola Pulsar II IMTS system and a Motorola Mostar radiotelephone. The performance of these systems was less than satisfactory. Contributing factors to the poor performance included: topography, atmospherics, antenna height, and antenna location. The Pulsar II system is designed to perform as a regular land line telephone, but rarely did. Most of the time out-going calls had to be placed through a mobile operator. In-coming calls were nearly impossible to receive. The Mostar is a microphone set that requires mobile operator assistance on all but local calls. Out-going and in-coming calls were generally successful with the Mostar, but unlike the Pulsar II system, had an electronically controlled time limit. Operationally, the Pulsar II system was used to place out-going calls and the Mostar to receive in-coming calls.

All communication equipment was leased for the duration of the project. The monthly equipment charges for the equipment were $2350. In addition, initial installation charges of $3,500 and two base antennas totaling $1,000 were incurred. Monthly call charges averaged $700 per month during the course of the field program.

The communication systems were invaluable in maintaining a safe and efficient field operation. From a safety perspective, all field personnel (rig geologists, engineers, field geologists) were required to have a radio at all time to enable reporting of injuries or other safety related problems. At the beginning of each day, field personnel would call in reporting their departure from the base camp and their arrival at their assigned work station. Likewise at the end of the day, personnel called in reporting their departure time, route of travel, and arrival at the base camp. A log of radio traffic was maintained at the base camp field operations office. Efficiency of the field operation was greatly enhanced by the radios; allowing consultation between the rig geologists and the field manager and project engineer concerning geologic and engineering conditions encountered during the drilling. As a matter of practice, the rig geologist would call in upon reaching the planned total depth of a boring and review the conditions encountered with the field manager before terminating the boring. Geotechnical requirements resulting from design revisions were expedited as a result of direct communication with the Redmond Office and WSDOT headquarters.

Health and Safety

A site specific Health & Safety plan was developed and issued to all project personnel. Prior to mobilization to the field, previous arrangements had been made with Airlift Northwest, located in Seattle, to medivac any serious injuries from the project site. A permanent, well marked helipad was maintained at the camp if the need ever arose. Fortunately, the Emergency Medical System never had to be activated.

During the course of the project, some concern was raised that the debris dam forming Castle Lake could possibly breach sending a torrent of water down the Toutle River valley toward Camp Baker, the location of the base camp. Golder Associates senior personnel inspected the debris dam and conducted a stability analysis which concluded that a catastrophic failure of the debris dam was a remote possibility in the event of seismic shaking. Contact was established and maintained with the United States Geological Survey's (USGS) David A. Johnston Cascades Volcanoe Observatory located in Vancouver, Washington. The USGS maintained a telemetry early warning system and sirens within the Toutle River valley and would notify the base camp at Camp Baker via radiotelephone if an event should occur. To facilitate camp evacuation if an earthquake occurred, an emergency escape route was cleared to high ground through the second growth timber growing along the slope behind the camp. The escape route was clearly signed and illuminated at night.

Personnel and Scheduling

A project staffing schedule was devised which provided timely completion of the project and yet still offered field personnel adequate time off. Field personnel at any one time usually consisted of the project engineer, project geologist, field manager, geologist and engineer to review drill logs and samples, lab technician, four rig geologists, and field secretary. Each drill was staffed full-time by a professional geologist to maximize the quality of collected drillhole data and to accurately record the typically variable geologic conditions. In addition, all drill rig geologists attended a one day indoctrination and field trip led by Golder Associates senior staff to familiarize the rig geologists with the local geology. All field personnel worked on a 10 days on-site and 5 days off-site rotation. Schedules were arranged such that 2 rig geologists rotated in and two out every five days. Transportation to and from the site to Golder Associates' office in Redmond, Washington was provided by a shuttle supplied by a WBE subcontractor (Halme Concrete Sawing). In addition, the shuttle transported supplies, samples, and equipment to and from the site to the main office and the MBE laboratory (Hong Consulting Engineers).

Daily Work Flow

At the beginning of each day, the rig geologists picked up drill hole prognosis forms, commonly referred to as "progs" for the anticipated hole conditions that individual would be logging that day. The "progs" were previously prepared the night before by one of the geologists or engineers. The "progs" contained the following information:

- Hole number, stationing and offset
- Anticipated depth
- Expected Stratigraphy
- Sampling schedule
- Piezometer Requirements
- Purpose of hole for design, i.e., cut, fill, retaining wall, bridge pier/abutment, etc.
- A cross section sketch showing the location of the hole relative to the design element.

The "progs" were quite useful for decision making when unanticipated conditions were encountered and minimized unnecessary drilling while insuring that all necessary geotechnical data was collected.

At the conclusion of each day, the field drill log and samples were transported to the lab trailer at Camp Baker. After receipt of the samples and field logs, the drill hole logs would be reviewed by a geologist/engineer for consistency and accuracy. Upon review, the field log would be submitted to the project secretary for entry into GOLDLOG, a proprietary borehole logging software package which produces report quality logs. Draft copies of the logs would be submitted for review, corrections made, and a final report quality log printed on the on-site laser printer.

The project engineer would review the log and develop a geotechnical testing schedule. Samples to be tested were either tested on-site or shipped to Hong Testing.

As geotechnical data became available, draft design reports were written in the field with the aid of computer based word processing software. As the need dictated, a draftsman from the Redmond office was brought to the site to produce necessary drawings. Once a week, the Golder Associates design team met with the WSDOT project managers on-site to discuss design issues, difficulties encountered, environmental and safety issues, project budget, and schedule. These on-site weekly meetings provided an excellent forum for the rapid resolution of problems and design issues.

At the conclusion of each day, the field manager agreed on daily contract quantities with all on-site subcontractors. This contract information was entered into an on-site computer spreadsheet which allowed daily tracking of all field costs.

SUMMARY

The Spirit Lake Memorial Highway field investigation provided some unique challenges that required out-of-the-ordinary solutions. Scheduling requirements by the WSDOT required a large magnitude project to be completed in a short time frame and within the fixed budget. Adding to these already formidable requirements was the conducting of a major field investigation in mountainous terrain during the winter months and within the blast zone of an active volcano.

To meet these requirements and challenges it was imperative to clearly establish what geotechnical data was required and how it would be obtained in the most cost and time effective manner. Considerable effort was expended "up-front" in defining these requirements through extensive field reconnaissance and the combined corporate experience of Golder Associates' senior staff.

Paramount to meeting the schedule and budget requirements of the project was a qualified drilling contractor who supplied the equipment and personnel to do the job and was able to adapt and deal with unusual conditions and circumstances.

The remote base camp was a cornerstone to the successful completion of field investigation. The base camp provided a safer and more cost effective project by eliminating long commuting distances to-and-from the site during winter months. As a result of eliminating long commuting time, more productive on-site work time was realized. The camp and on-site office facilities allowed on-site engineering, laboratory testing, project management, contract administration, and closer coordination with the WSDOT.

Key to the efficient operation of the field program was a single, full-time, experienced field manager to coordinate the operational needs of the project and the daily management of schedule and costs. This on-site management provided real-time tracking of budget and progress against the project objectives allowing rapid responses to changing conditions. As a result Golder Associates completed the filed investigation ahead of schedule and under budget.

ACKNOWLEDGEMENT

The authors would like to thank Mr. Steve Lowell of the Washington State Department of Transportation for providing Golder Associates Inc. the opportunity to participate in the Spirit Lake Memorial Highway project.

ANALYSIS AND DESIGN OF TIE-BACK WALL NO. 5 IN STEUBENVILLE, OHIO

by: Richard W. Humphries[1]
Gordon M. Elliott[1]
John Hollenbaugh[2]
Gerald Cafarelli[2]
Gene Geiger[3]

ABSTRACT

One of the largest tie-back wall projects in the world is under construction by the Ohio Department of Transportation in Steubenville, Ohio. Tie-Back Wall No. 5 is the largest of 7 tie-back walls being constructed as a part of the U.S. 22/S.R. 7 interchange to connect with West Virginia's new Weirton-Steubenville bridge over the Ohio River. This wall will be constructed in four tiers, with a maximum height of 130 feet and a length of 2,840 feet. Construction of the wall will require 2,140 tie-back anchors with capacities up to 245 kips and free lengths up to 115 feet. The excavation for the wall is in horizontally bedded sandstones, shales and coals of Pennsylvanian Age. By incorporating a pressure relief tunnel behind the wall, it has been possible to reduce the tie-back requirements by approximately 50%. This represents an estimated saving to the project of $5.5m.

This paper describes the overall project, the investigations, the potential modes of failure, the stability analyses and the design of the wall, the tie-backs and the pressure relief tunnel system.

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[1]Golder Associates Inc., Atlanta, GA

[2]URS Consultants, Inc., Akron, OH

[3]Ohio Department of Transportation, Columbus, OH

INTRODUCTION

The Ohio Department of Transportation (ODOT) is beginning construction for the approaches to West Virginia's new cable-stay bridge for U.S. 22 over the Ohio River near Steubenville (Figure 1). The Construction Contract includes seven tie-back walls, three bridges, a tunnel and the construction of approximately four miles of roadway. The layout of the project is shown in Figure 2. In the vicinity of Wall No. 5 the existing natural slope above the west bank of the Ohio River is steep, with slope angles up to 45 degrees, slope heights of 350 feet and thin soil cover. The eastbound ramp of SR-7 onto the new bridge requires a large excavation at the toe of this slope. A tie-back wall has been chosen to support the excavation and to preserve Steubenville College which is located at the top of the slope above the wall. A profile of the wall is shown in Figure 3.

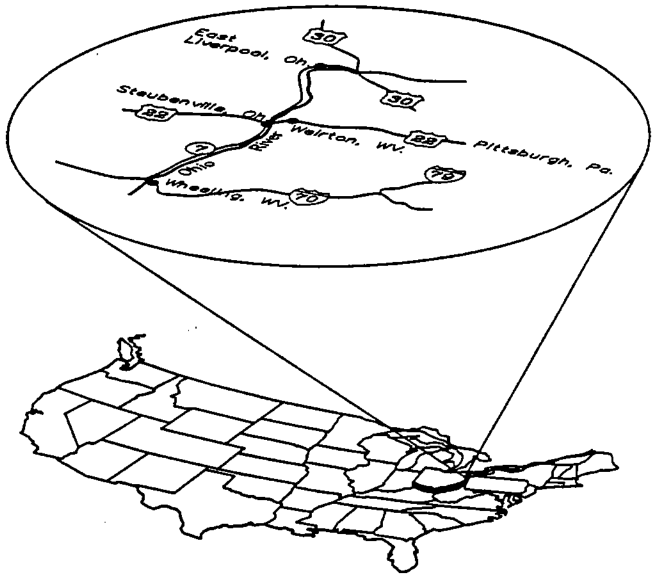

FIGURE 1: MAP LOCATION SITE

The design of the wall was addressed in four stages. First, the geologic and hydrogeologic setting was characterized and potential modes of instability were identified. Second, a method of analysis was developed, geotechnical parameters were estimated and tieback anchorage design curves were developed for drained and undrained conditions. Third, a hydrogeologic analysis was carried out to design a pressure relief tunnel system that would lower water levels to those assumed by drained conditions. Last, the detailed designs of the tie-backs, the wall facing and the pressure relief tunnel system were developed.

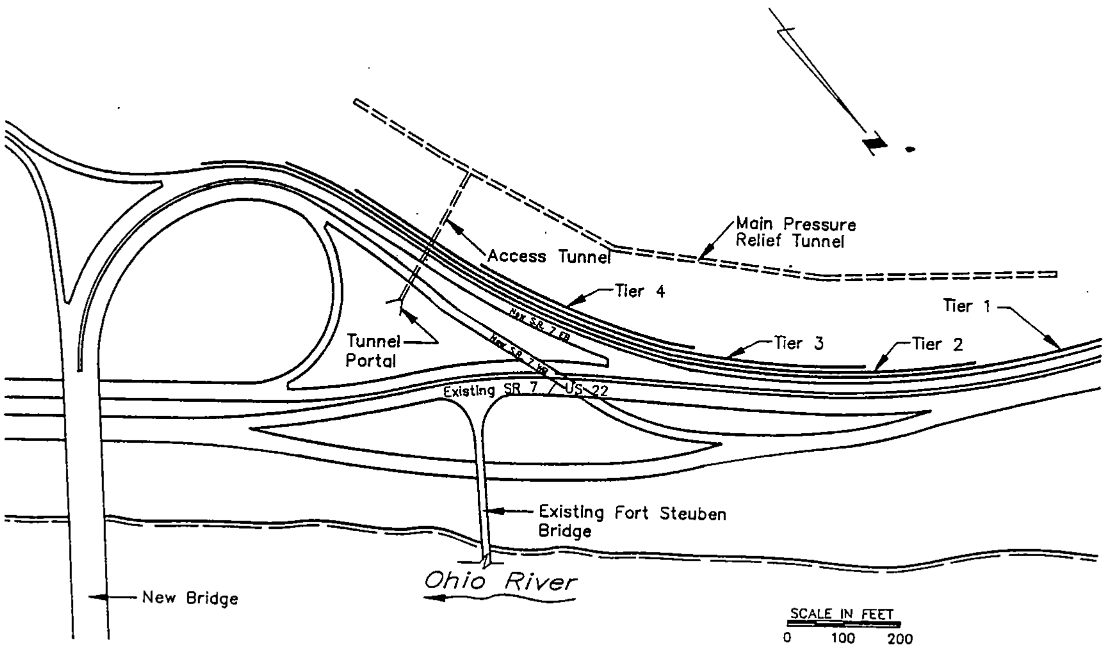

Figure 2: PLAN OF TIE-BACK WALL NO. 5 & PRESSURE RELIEF TUNNEL SYSTEM

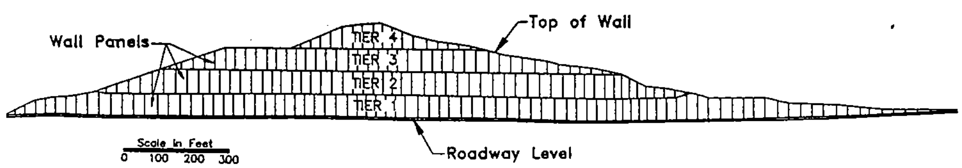

FIGURE 3: PROFILE OF TIE-BACK WALL NO. 5

GEOLOGIC SETTING

The project area is within the Appalachian Plateau physiographic province, which is characterized by a thick, flat-lying sequence of Pennsylvanian-age sandstones and shales, with minor coal seams and limestone beds of the Conemaugh series. The area is just south of the farthest extent of glaciation during the Pleistocene Epoch. Even though the rocks are on the order of 300 million years old, they have experienced little in the way of stress, either from ice loading, deep burial or tectonic activity. Consequently they exhibit infrequent fault and joint structure.

A geologic cross-section through the hillside at the site is presented in Figure 4. The sandstone units are generally massive though they vary in consistency across the project. The shale units vary significantly. The grey shales tend to be massive, whereas the red shale units have well defined bedding partings and some slickensided joints. Several of the dark grey shale units have sandstone, siltstone and coal interbeds. Some of the coal units are associated with a variable underclay layer.

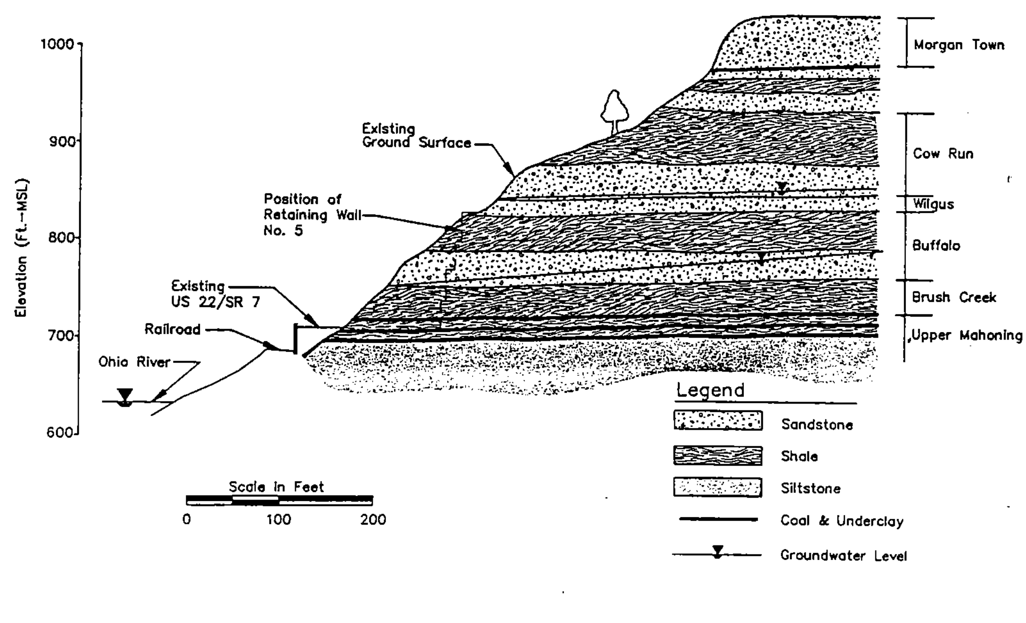

FIGURE 4: GEOLOGIC SECTION THROUGH EXISTING SLOPE

The "red-shale" units of the Conemaugh Series are noteworthy because they have been associated with slope failures in southeastern Ohio (Wu et al.; Gray et al.; Fisher et al.).

At Wall No. 5 there are two hematite-stained shale horizons approximately 110 feet and 0 to 10 feet above the base of the wall. The lower of these two horizons underlies a coal seam and associated underclay of varying thickness and consistency. The strength of this zone of red shale, coal and underclay, referred to as the "Upper Mahoning Shale", is most likely to control the stability of the hillside .

The significant rock structure at the site is the horizontal bedding and three sub-vertical joint sets. The most prominent joint set strikes parallel to the hillside and dips towards the river at between 70 and 80 degrees. It is related to tension-stress patterns formed as the Ohio River incised its canyon and is persistent through all geologic units. The other two joints strike approximately 45 degrees to the hillside and dip between 70 and 80 degrees towards the south east. These joint sets are only visible in the more rigid, sandstone units.

Ground water exits the face of the existing slope at the base of each of the sandstone units. The low permeability shales and underclay units between the sandstone units form aquitards in the hydrogeologic setting. The water levels in piezometers installed in the exploratory coreholes confirmed that there is a base water table and two or three perched water tables in the stratigraphic section. The base water table is hydraulically connected with the Ohio River. The water levels in two of the aquifers at the site are shown in Figure 4.

To analyze the expected loading on the retaining wall a number of postulated mechanisms for rock movement were examined. Deep-seated rock slides are relatively rare in the eastern and south-eastern Ohio. Where such slides have occurred, large blocks of rock are reported to have separated from the valley walls along near-vertical stress relief joints and slid along weak claystone or shale beds. This mechanism is certainly supported by the geologic information available for the site. Water pressures in the slopes are usually significant contributing factors to such slides.

Other modes of rock movement, involving circular and curvilinear failure surfaces, are not supported by the available geologic evidence, and when analyzed gave less critical factors of safety in a limit equilibrium analysis.

GEOTECHNICAL ANALYSIS

The analysis of sliding blocks was addressed using limit equilibrium methods in which the forces causing the block to slide were compared with the forces opposing movement. Given the rock structure at the site, an unstable block is most likely to slide out of the hillside on a plane through the Upper Mahoning shale, with the release surface provided by a stress relief joint. The stress relief joints were observed to be ubiquitous at the site, so the size of the sliding block could not be bounded by field data.

Instead, the geotechnical analysis focused on identifying the amount of tie-back force that would be required to prevent a block of a given size from sliding out of the hillside. By this method it is also possible to identify the smallest block that would just remain in place in the hillside if no tie-back force is provided.

The forces causing the block to slide are provided by the down slope component of the weight of the sliding block acting along the sliding plane, and the hydrostatic pressures imposed by the ground water on the release surface and the sliding surfaces of the block. The weight of the sliding block and the inclination of the plane of sliding are defined by the geometry of the slope, the excavation and the geology. The hydrostatic pressures on the release surface were estimated from measured ground water levels. The shape of the sliding blocks and the imposed loads are shown schematically in Figure 5.

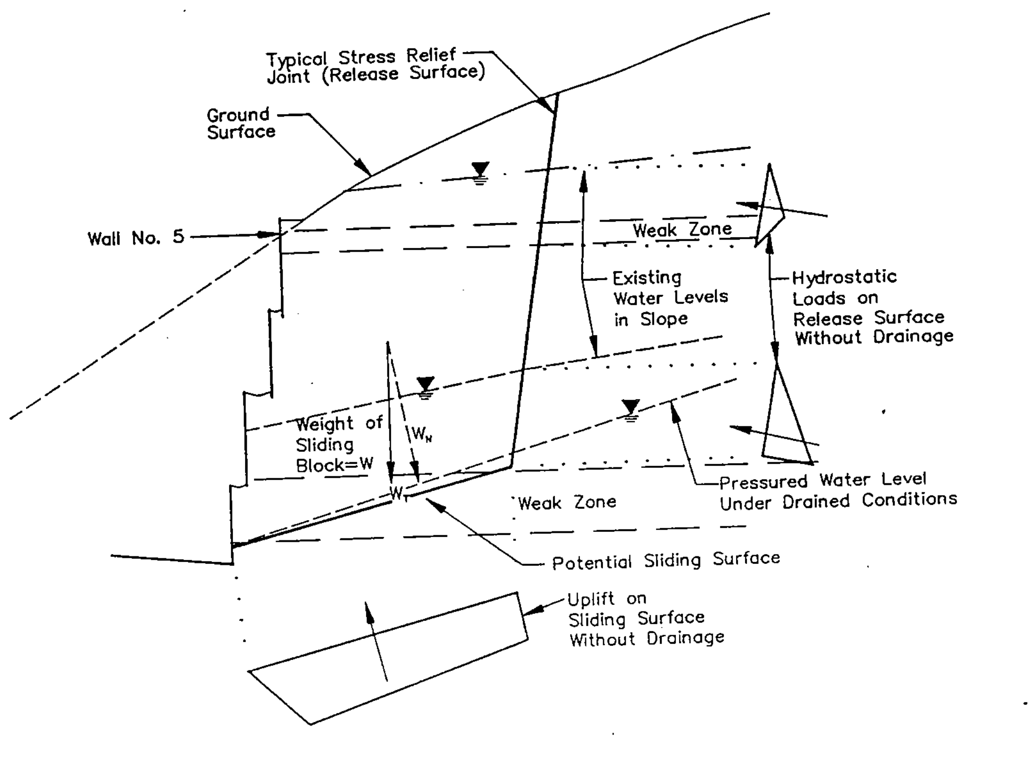

FIGURE 5: TYPICAL LOADING ON SLIDING BLOCK

The forces opposing sliding are provided by the frictional strength of the material forming the base of the block and the force provided by the tie-backs. The strength of the material forming the base of the block (the Upper Mahoning shale) was estimated through a program of laboratory shear tests on saturated samples collected during the field investigation, and through back-analysis of the existing slopes. On the basis of these studies, a Mohr-Coulomb strength criterion, described by a friction angle of 17 degrees and a cohesion of zero, was adopted for the analysis. In calculating the frictional component of the force opposing sliding, the normal component of the weight of the block was reduced by the groundwater uplift pressure of the groundwater on the sliding plane.

The design tie-back force was calculated to give a factor of safety (ratio of restraining forces to driving forces) of 1.5 under long-term static conditions. The design was also checked to ensure a factor of safety that exceeded 1.0 for both immediate post-construction conditions (undrained) and seismic conditions. Tie-backs were designed to be inclined between 10 and 25 degrees down from horizontal for ease of grouting and to provide a force directed approximately normal to potential release surfaces. The results for different sized blocks are illustrated in the form of a graph of the required tie-back force per foot run of wall plotted against the free length of the tie-backs (Figure 6). Design curves of this form were prepared for critical sections along the length of the wall.

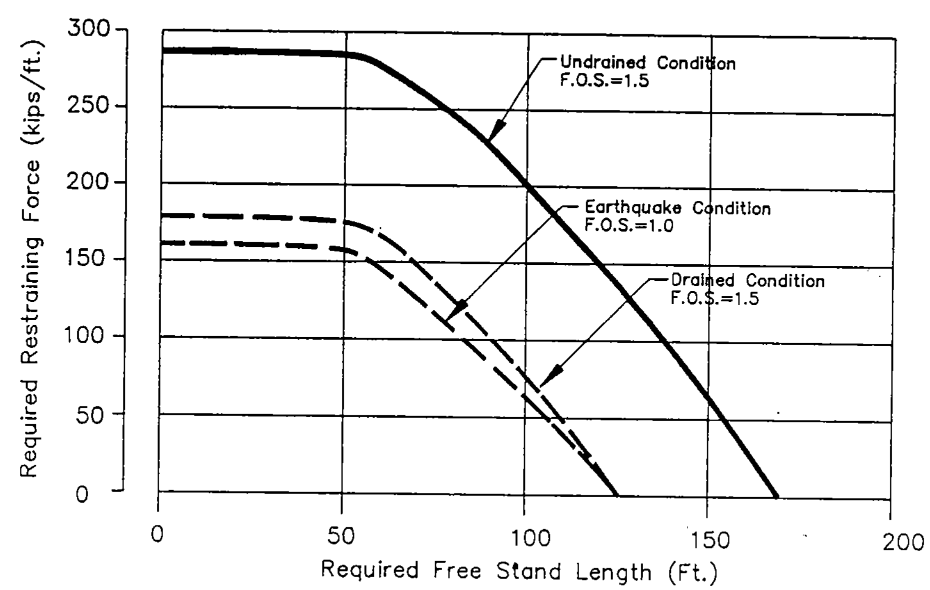

FIGURE 6: DESIGN CURVE

On the basis of a preliminary tie-back layout designed to satisfy these geotechnical requirements, a rough estimate of the cost of the tie-back anchors alone would exceed $16m. Therefore, a means of reducing costs was sought. Since hydrostatic loads present the major contribution to the destabilizing forces, a series of stability analyses were carried out to investigate the effect of lowering the ground water. The results of these analyses indicated that the total tie-back force could be reduced by 50% (area under the design curve reduced by one-half as illustrated for the drained condition curve in Figure 6) if the aquifer could be drained to below the assumed level.

HYDROGEOLOGIC ANALYSIS

The pressure relief tunnel system was designed to effect this lowering of ground water levels in the lower aquifer. The lower aquifer system was modelled using a two-dimensional analysis of a uniform, homogeneous, transverse-isotropic medium. An upgradient constant head boundary condition was assumed 500 feet behind the foot of the wall. A free flow boundary condition was assumed at the free face of the hillside. The ground water levels defined the upper flow boundary, and the aquifer was assumed to have infinite extent below the ground water surface. Estimates of horizontal permeability of the rock units forming the aquifer were derived from measurements that had been made during the field investigation. Estimates of the vertical permeability and effective porosity used in the analysis were based on experience of similar rock types. The model and the selection of input parameter values was validated using existing ground water observations.

The ground water model was then used to identify the optimum configuration of the pressure relief tunnel system. This, combined with an evaluation of geologic conditions, resulted in a design which incorporates a 10 feet x 10 feet main pressure relief tunnel, 200 feet behind and below the base of the wall, combined with a drainage curtain of open boreholes drilled into the tunnel crown along the tunnel axis. A section through the tunnel, showing the local geology is presented in Figure 7. The main pressure relief tunnel will be over 1,950 feet in length and will be oriented parallel to the central portion of the wall. Access to this main tunnel will be by an access tunnel driven directly into the hillside beneath the roadway. The layout of the main tunnel and access tunnel is shown in Figure 4. The grades of both tunnels have been designed primarily to ensure that there will be a minimum of 10 feet of competent rock above the crown, and also to allow water in the tunnel to drain by gravity to the portal.

The drainage curtain will consist of a series of 2 inch diameter, 85 feet long parallel open boreholes drilled into the crown of the main tunnel at 20-foot centers along the tunnel axis. A program of in situ monitoring will be used to confirm that the drainage curtain does effectively lower ground water below the level assumed for the design of the tie-backs. Prior to drilling the drainage curtain, piezometers will be installed in holes drilled upwards from the main tunnel at 200-foot spacings to

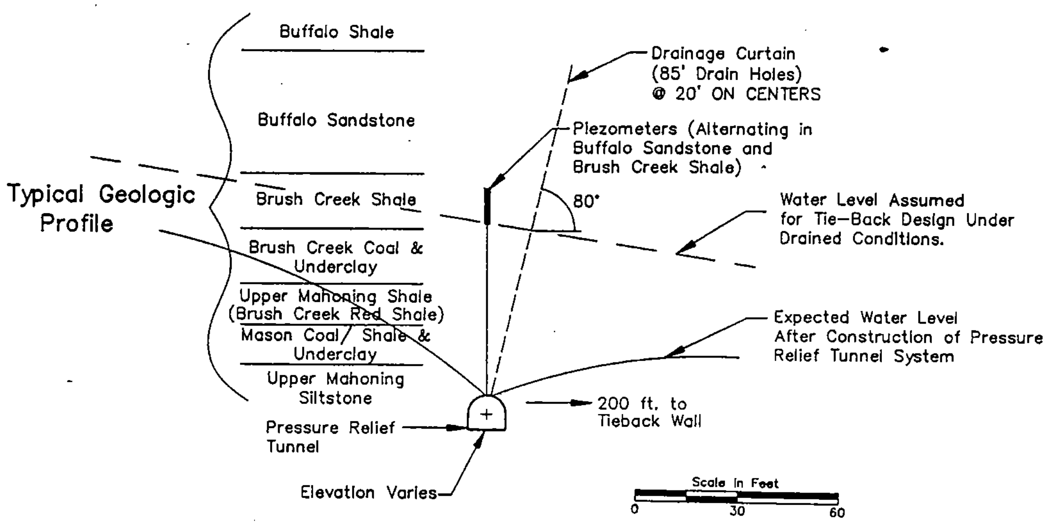

FIGURE 7: SECTION THROUGH PRESSURE RELIEF TUNNEL

monitor hydrostatic pressures in different overlying strata, as shown in Figure 7. These piezometers will monitor the reduction in hydrostatic pressure in overlying strata as the drainage curtain is drilled. If, upon completion of the drainage curtain, hydrostatic pressures in overlying strata have not responded sufficiently, then a set of secondary drain holes will be drilled in the same plane to split-space the primary drain holes. If necessary, tertiary drain holes split-spacing the primary and secondary holes will be drilled to ensure that ground water is lowered below the level assumed in the design. Ground water inflows during excavation of the tunnel and drain hole drilling are expected to be low. They should reduce further with time.

An estimate of the cost of constructing the pressure relief tunnel system is $2.5m, and will result in a savings in tie-backs of approximately $8m.

DESIGN AND CONSTRUCTION DETAILS

Tie-Back Wall

The design curves provided by the geotechnical analyses were used to form the basis of the layout and design of the tie-backs used for maintaining the stability of the hillside as the toe is excavated. The length of the tie-backs varies to suit the required anchorage forces at different points behind the wall. The maximum free stressed length for a tie-back is 115 feet, and the maximum working load of the anchors is 245 kips. The total number of tie-backs in the final design amounts to 2140. A section through the wall showing the tie-back layout is presented in Figure 8. Tie-backs will be inclined 10 degrees down from horizontal except in the lower sections of the wall where the angle is set to 25 degrees to avoid anchoring the tie-backs in the weak Upper Mahoning shale. The bond length for each anchor will be assessed on the basis of field testing.

Excavation for the wall will be from the top down with tie-back anchors being installed and pre-stressed as the excavation proceeds. The wall will be constructed in four tiers.

In the overburden and highly-weathered rock, soldier piles will be spaced at 8 feet or 10 feet 8 inch centers. Timber lagging or shotcrete will be used for temporary support of the vertically excavated face. In these areas the load from the tie-backs will be applied to the soldier piles. The maximum vertical spacing of the tie-back anchors will be 9 feet.

In sound rock, the face will be excavated vertically and the loads from the tie-backs will be applied to the rock either by bearing plates or by pipe casings grouted in place to provide load transfer through shear. The tie-backs will be spaced horizontally at 8 feet, 10 feet 8 inch, or 16 feet centers and vertically at 10 feet maximum centers. The concrete facing is secured to the rock face by 5 feet x 5 feet maximum patterns of No. 9 rock dowels.

The reinforced concrete facing of the wall will be 12 inches thick, and will be constructed as a series of panels. Each panel will have a specific number of tie-back anchors to suit the overall tie-back force required for stability of the hillside. The number of tie-backs per panel varies up to a maximum of 16.

Wall movements will be monitored during and after construction with electronic distance measuring equipment and prisms installed on the concrete facing of each tier.

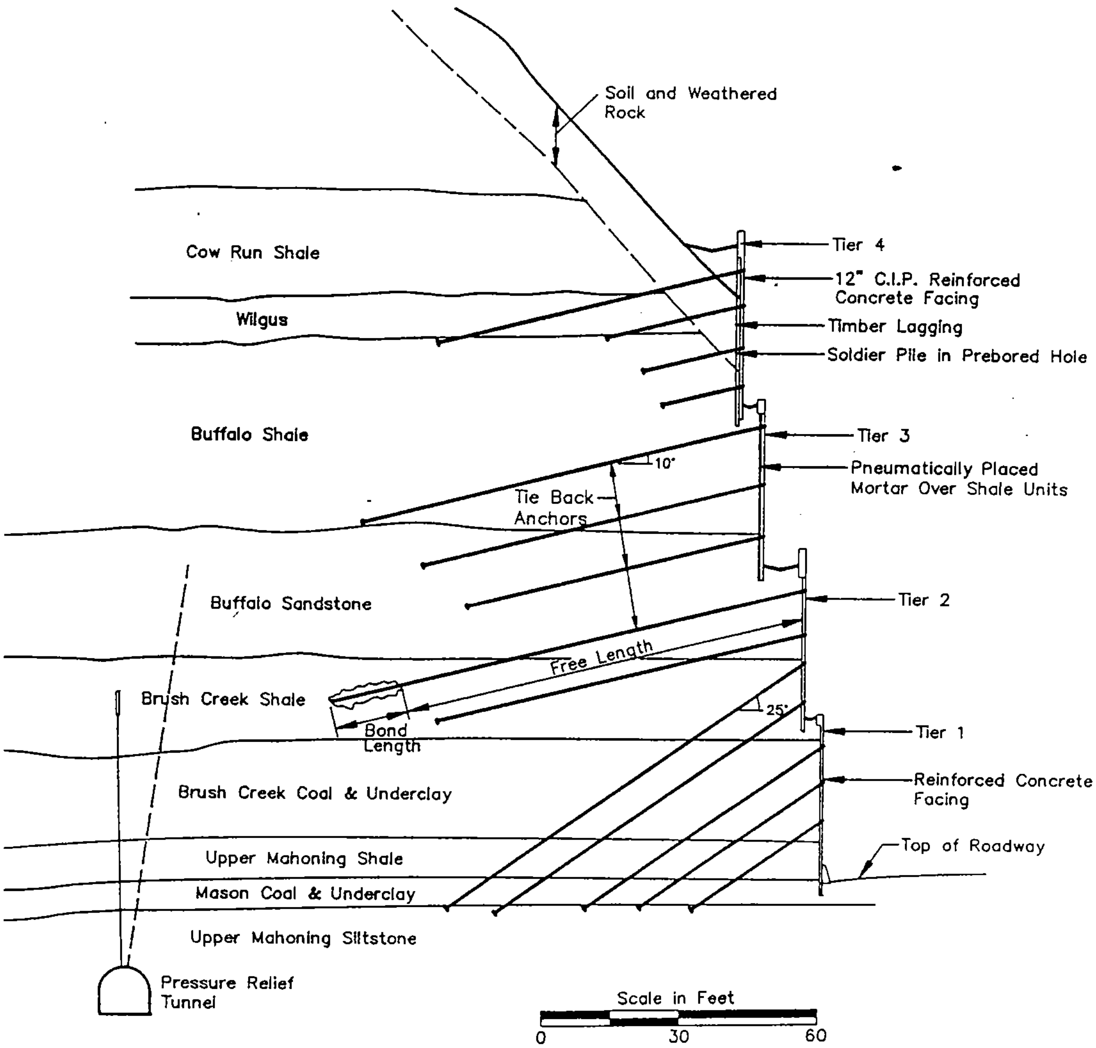

FIGURE 8: SECTION THROUGH TIE-BACK WALL NO. 5

Pressure Relief Tunnel System

Contractors have been given the option of excavating the tunnels by drill-and-blast, roadheader or by tunnel boring machine. Excavation of the pressure relief tunnel system will start at the same time as the excavation of the wall, but is required to be completed before half of the wall height has been excavated. Ground support forthe tunnels will comprise pattern bolting with shotcrete except in limited zones of poor rock where lattice girders, spiles and shotcrete may be required.

The first 160 feet of the access tunnel will be lined with reinforced concrete due to reduced cover and U.S. 22 traffic loads. Construction will start in Summer, 1990. The pressure relief tunnel system is expected to take approximately nine months and Retaining Wall No. 5 should take approximately 3 years to complete.

ACKNOWLEDGEMENTS

The authors of the paper wish to thank the Ohio Department of Transportation for permission to publish this paper. URS Consultants, Inc. are the prime design consultants for the project. Golder Associates Inc. acted as geotechnical sub-consultants for the development of the tie-back anchorage requirements and the design of the pressure relief tunnel system. Resource International were geotechnical sub-consultants for the site investigation.

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ALTERNATIVE METHODS FOR RETAINING WALLS

Peter Nicholson and Spark Johnston

Nicholson Construction Company

INTRODUCTION

Over the last 20 years, there have been a series of developments that have had a major impact on the design and construction of earth and rock retaining structures. This has not been confined to just new construction but has also impacted our methods of dealing with slope stability and landslides on existing facilities. For many state highway departments, the conventional cantilevered concrete retaining wall is a thing of the past in all but the smaller, low height structures. Costs of right-of-way acquisition and construction, as well as the increasing constraints imposed by environmental impact have made laying back the slope and rock buttresses less practical. Permanently anchored walls, rock bolting, soil nailing, reinforced shotcrete and other methods, while not all new, have been improved and refined and are being used more frequently.

To give a better appreciation of the issues involved, the discussion will be separated into three prime categories:

1. Specification and Contractual
2. Retaining structures for new construction
3. Remedial work for existing slides or slopes

The following discussion should give the reader an overall view of what has been done in the past, what is recent or emerging technology, and what the future might hold.

I. Specification and Contractual

IN ORDER TO ALLOW ADVANCES IN TECHNOLOGY TO BE INTRODUCED IN THE PUBLIC SECTOR, CONSTRUCTION MARKETING, NEW CONTRACTING AND SPECIFYING METHODS WILL HAVE TO BE USED.

This may sound like an alarming statement to those who have been used to our present system of:

1. Owner identify the project

2. In-house or consultant design

3. Low-bid contractor build as per plans and specs.

If you stop and think about it, however, we are already using variations on this standard scenario in almost every project. All highway projects with federal money now must have an FHWA mandated Value Engineering clause. Many of our bridge projects have alternate designs and some call for contractor alternate design. Many states have adopted alternative bidding for mechanically stabilized walls. These procedures have been successful in saving millions of dollars for the taxpayer over the years.

It is interesting to note that most Europeans and Japanese practices vary considerably from ours. They are interested in "end results" and the way to get there is developed by a team consisting of the owner, his consultant and the contractor. The contract documents don't take a "cookbook" approach, instead they permit any solution which will yield the intended results. Quality is insured by:

1. References that demonstrate success on previous projects

2. Durability is secured by appropriate guarantees

3. Non-destructive tests confirm results in the field.

These measures have allowed Europe and Japan to be 5-10 years ahead of the United States in applying technological innovation to construction processes.

The reasons that alternative contracting and specifying procedures are necessary for innovative technologies to be used are complex. We recognize that there is a natural tendency in all of us to get complacent and comfortable with those things that we know. There is less hassle and fewer questions to be answered, nobody will look over your shoulder and second-guess you about why the choice of conventional solution, whereas the choice of an anchored wall may be more economical but will be questioned by your associates.

In order to overcome these natural instincts we must provide incentives. In our present system there is little incentive for the engineer or geologist to come up with something new, and there is no easy way for him to learn in depth about new techniques. Most contractors still are only comfortable with performing work to plans and specifications developed by someone else. They feel there is increased liability and not enough reward to change their way of doing business. However, there is a growing body of owners, engineers and contractors who are not satisfied with the status quo, who realize that we are in a competitive race with the rest of the world to be the most efficient and most productive. These people are making their voices heard. It is my belief that new technology is here to stay and that if we as professionals don't keep up, we will all be the losers.

Some variations of the standard U.S. procedures that provide incentives and allow the rapid introduction of new technology have been used by some states to achieve even greater economies where the owner realizes all the savings up front. The two most common methods are contractor pre-bid design and post-bid design.

In order to use these methods, the owner must identify his needs and should have an idea about what type of alternatives may be proposed. The specification should then give the design parameters to be used as well as constraints regarding right-of-way, corrosion protection, and other similar details. These two methods are described briefly below.

METHOD 1 - PRE-BID DESIGN

The contract documents in this method are prepared to allow for various retaining wall alternatives. The typical wall alternatives can include the standard cantilevered retaining wall or gravity wall, the permanently anchored wall, the Reinforced Earth Wall or the Doublewall or combinations of other wall types. The owner/designer provides a conceptual type, size and location of the required wall. In addition, prequalification requirements, geotechnical reports and data, design parameters, site limitations (e.g. right-of-way limitations), design submittal requirements, material requirements, instrumentation requirements, testing requirements, acceptance criteria and method of payment are provided for the anchor contractor's use. This information is usually made available 60-90 days prior to the bid date. Prequalified specialty contractors prepare and submit final design calculations and construction drawings for the owner/engineer's review. Once the designs are approved, the specialty contractors are placed on a list of "prequalified specialty contractors with approved designs". This list is then distributed to the general contractors. General contractors receive bids only from the specialty contractors with approved designs. Once a contract is awarded, construction begins.

METHOD 2 - POST-BID DESIGN

The contract documents in this method are also prepared to allow for the bidding of various retaining wall alternates. The major difference in this method is the timing of the design and the bidding. The owner/designer provides a conceptual type, size and location of the requested wall. In addition, prequalification requirements, geotechnical reports and data, design parameters, design submittal requirements, material requirements, instrumentation requirements, anchor testing requirements, acceptance criteria and method of payment are provided for the anchor contractor's use. During the bidding process the specialty contractor prepares a preliminary design and firm cost estimate. General contractors receive bids from prequalified specialty contractors for the various wall alternates and subsequently select the lowest price to include in their bid. Once the contract has been awarded, the specialty contractor prepares his final design and construction drawings and submits the documents to the owner/designer for review. After acceptance of the design, construction begins.

METHOD 1 vs. METHOD 2

The pre-bid design method allows good technical input and cost effective contractor selection. The main drawback can be in the review system used to prequalify contractors. The review group should contain an engineer familiar with permanent anchor design and construction, a structural engineer, and the owner's foundation engineer.

The post-bid design also has some drawbacks. Although similar to the pre-bid design method, the post-bid method often results in the general contractor "shopping" for marginal and/or unqualified subcontractors to install the anchors. Even though the specifications contain prequalifying language, seldom is this enforced once the contract is awarded. A recommended solution to the problem is to establish three lists of prequalified specialty anchor contractors and <u>limit</u> the "shopping" to the list.

II. Retaining Structures for New Construction

The use of the two methods described above by both the FHWA and by various state highway departments was begun about 10 years ago in several states. The results are almost uniformly positive. Large economies have been achieved and attractive, long lasting structures have resulted. These permanently anchored walls are no longer in the demonstration or experimental phase, they are standard practice in many states and their cost relative to conventional construction continues to decline. Some examples are shown below:

A. Wall 14, Atlanta, GA

Site constraints precluded a conventional permanently anchored wall because of right-of-way and a recalcitrant adjacent property owner. A conventional cantilevered wall was designed, but during the course of construction it became apparent that this design could not work without unacceptable movements that would endanger the existing structure. Nicholson Construction, working closely with the general contractor and the state highway department, proposed a novel solution that was accepted.

Since no encroachment could be made beneath the adjacent property and since there was only approximately 5 feet

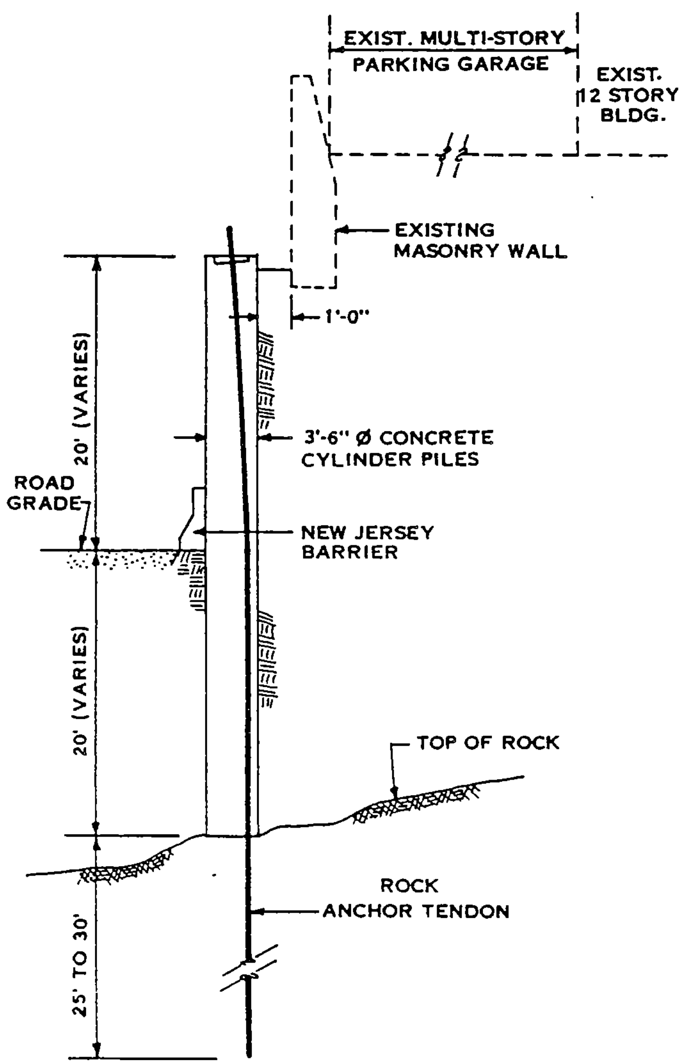

Figure 1. Post-Tensioned Caisson Retaining Wall

available from the face of the new structure to the property line, space was at a premium. The solution is shown on Figure 1. The vertical draped rock anchor that passes through the concrete cylinder piles provided the necessary lateral restraint for the wall by virtue of its eccentricity or off-set from centerline of 14 inches. The drape in the tendon helped provide the structural resistance bending necessary for the wall, much like a bridge beam, but rotated ninety degrees. Because of the small moment arm (14"), the anchor loads were high (700-870 kips). The performance of the wall during construction and after has been outstanding. During stressing of the anchor and prior to excavation the wall was pulled back toward the structure approximately 1/4". During and after excavation the outward movement of the wall was 3/8" for a net movement outward of 1/8" at the top of a 22 ft. high wall.

B. Ramp Q Abutment, Pittsburgh, PA

The first "new construction" anchored abutment used by Penn DOT was constructed on a depressed section of I-279 in 1987-88. The adjacent hillside was an ancient landslide and during construction of the new roadway, the hillside began moving. This movement resulted in a redesign of the retaining structures. Ramp Q abutment was in the middle of the affected zone and was approximately 30 feet high. The walls on both sides of the abutment were already designed as permanently anchored retaining walls and that design, with some modifications, was used as the abutment for the ramp. Given the assumed landslide generated soil pressures, construction of a conventional abutment would have resulted in a closing of an adjacent roadway and the use of a large temporary shoring wall. In addition, the amount of concrete required for the conventional pile-supported, gravity abutment would have been costly.

The overall layout of the abutment is shown in Figure 2. Since this was the first use by Penn DOT of an anchored abutment and was one of the first on the Interstate system, the FHWA and Penn DOT decided to heavily instrument the installation. Details of the instrumentation are shown in Figure 3. The overall deflection of the abutment after excavation was completed was on the order of 0.6", and the abutment has performed satisfactorily since being placed in service.

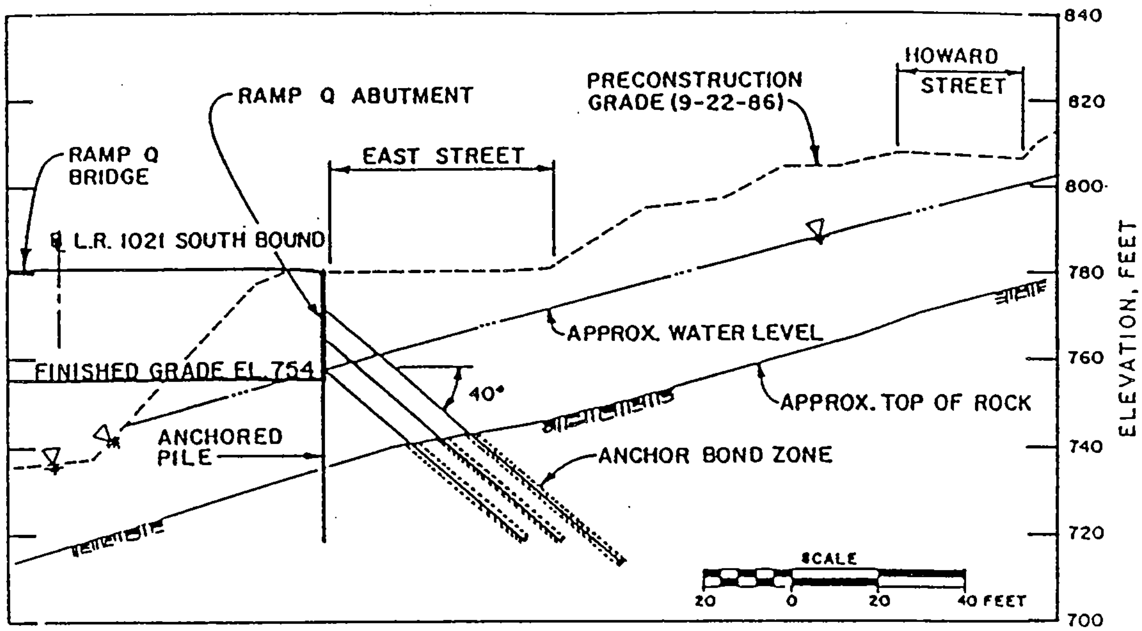

Figure 2. Schematic Cross-Section, Ramp Q Abutment

The abutment in this case was nothing more than an extension of the roadway retaining structures needed for support of the adjacent roadways and structures. The vertical cylinder piles had to be taken deeper than for the retaining walls so that the additional vertical load from the bridge could be safely carried, but there were no other major changes. The same architectural final facing was used and an aesthetically pleasing structure was constructed at an economical price in an active landslide.

III. Remedial Work for Existing Slides or Slopes

One of the greatest challenges most of us face as engineers and geologists is how to deal with existing slopes and walls that are sliding and failing. In most cases there are existing structures involving roadways, bridges, retaining walls and, in some cases, buildings that are threatened, moving, damaged or destroyed. For an existing slope or wall that begins to move after having been in service for more

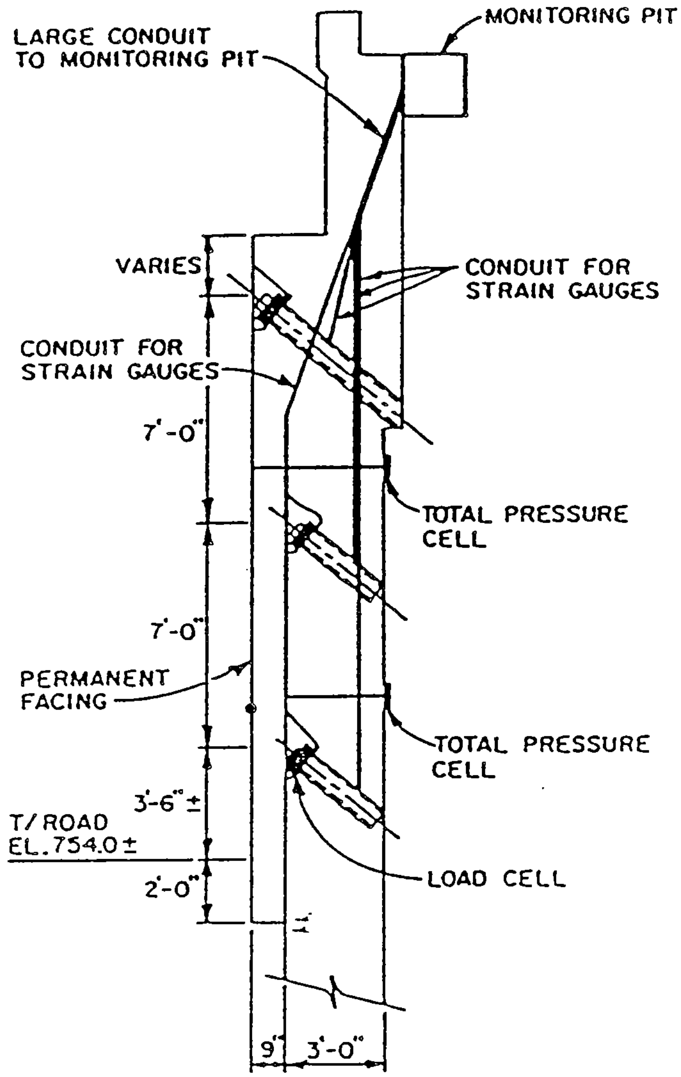

Figure 3. Instrumentation Details, Ramp Q Abutment

than a year, it is important for us to remember that at some point in time the slope or wall must have had a factor of safety of greater than 1. This says that what is in place still has strength. Digging at the toe and removing some of that strength in order to construct a fix can lead to move movement. If we can reinforce what is there already, we can usually solve the problem efficiently, with minimal disruption to existing facilities and at lower cost.

Again, one of the newer technologies comes into play here; Soil Nailing (also know as In Situ Reinforcing or INSERT[sm] walls. Again, the same issues from a specification and contractual point of view come into play - how to specify and contract for the work. And again, it is easier to take the conventional view and dig it out, or buttress it, but many time these are not the most cost effective solutions. Allowing alternatives to a conventional solution is one way to achieve the best results. A short discussion of two recent examples will show the benefits that can be obtained.

A. INSERT[SM] Wall-Type A, LR 69 Armstrong County, PA.

Legislative Route 69 runs along the Kiskiminetas River. Much of it is cut into the steep hillside along the bank of the river, with the cut material used as downhill side fill to provide the highway bench. In a number of places, this embankment material proved unstable and slides occurred - one such slide had reduced the roadway to one lane. A slide repair was designed by Penn DOT as an anchored caisson wall but with bids also accepted for alternative designs. The author's company was successful based on an alternate design (Figure 4). Rather than excavate in order to place the caissons and anchors, the in-situ soil was reinforced by an array of drilled and grouted Pin Piles that served to form a gravity wall.

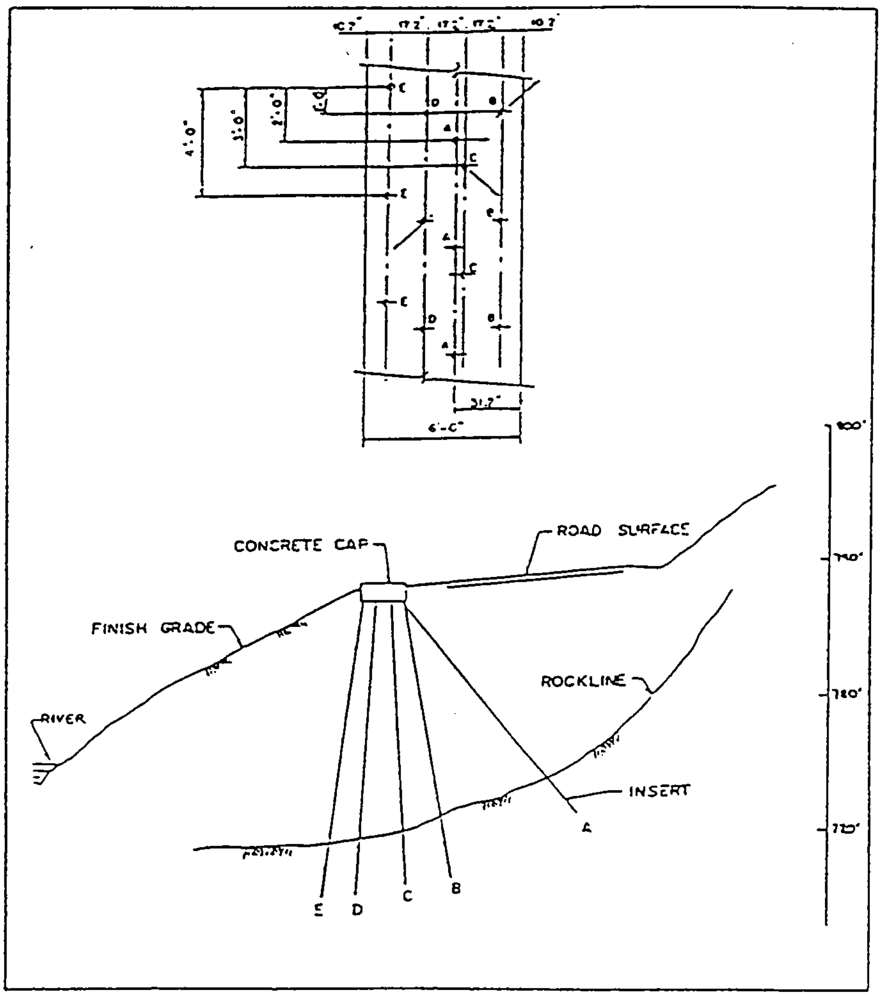

Figure 4. Details of LR69 In Situ Reinforcing - Type A Wall

The design height for the INSERT^{sm} wall was 34 feet and assumed a rapid drawdown of the river after a flood also acting on the wall. This necessitated the use of a tie-back (Row A of Figure 4) to stabilize the gravity wall formed by rows B, C, D, and E. All the inserts were keyed into rock a minimum of 15 feet and consisted of either a #11 or #14 rebar grouped into a 6" diameter hole. The spacing and orientation of the reinforcing members was chosen to force an interaction between the reinforcements and to insure that soil would not extrude between them. During the seven months prior to the start of the work, slope indicators showed a movement of about 0.5". Just prior to the start of the work, during a three month period, the movement accelerated to 3.75" with most of the movement at a depth of 30 ft. After the concrete cap was poured and approximately 40 of the reinforcing members installed in the most critical area, the movement appeared to have stopped. No subsequent movement of the roadway has been detected over the six years the wall has been in place.

This alternative design generated a savings to the state of about 15% on a $1+ million contract. The savings could have been greater and the wall still satisfactory by not designing for total scour of the soil in front of the wall and for rapid drawdown due to flood conditions at the same time. Approximately 1/2 of the reinforcing members could have been saved and perhaps a savings of another 30% in contract price.

B. Soil Nailing - Cumberland Gap, KY

This direct FHWA project was constructed to stabilize a slope near the Kentucky portal of a pilot tunnel under the Cumberland Gap. The parent rocks in the area are a series of shales, sandstones, and siltstones that have been differently weathered to form a soil mantle ranging in thickness from 5 to 20 feet. Beneath the soil overburden is weathered rock and, in general, sound bedrock is encountered at a depth of from 15 to 40 feet.

The design of the 40 foot high section of the wall resulted in a configuration as shown in Figure 5. The spacing of the nails was a 5x5 grid with an assumed nail diameter of .58 feet. The nails consisted of #8 and #11 grade 60 reinforcing bars grouped into a 4-1/2" diameter hole. Up to 8 rows of, and a total of 335 nails, were installed in the 9,000 square foot wall.

The wall was instrumented with three different types of load and movement measuring devices, slope indicators, strain gauges on the bars and electronic distance measuring points.

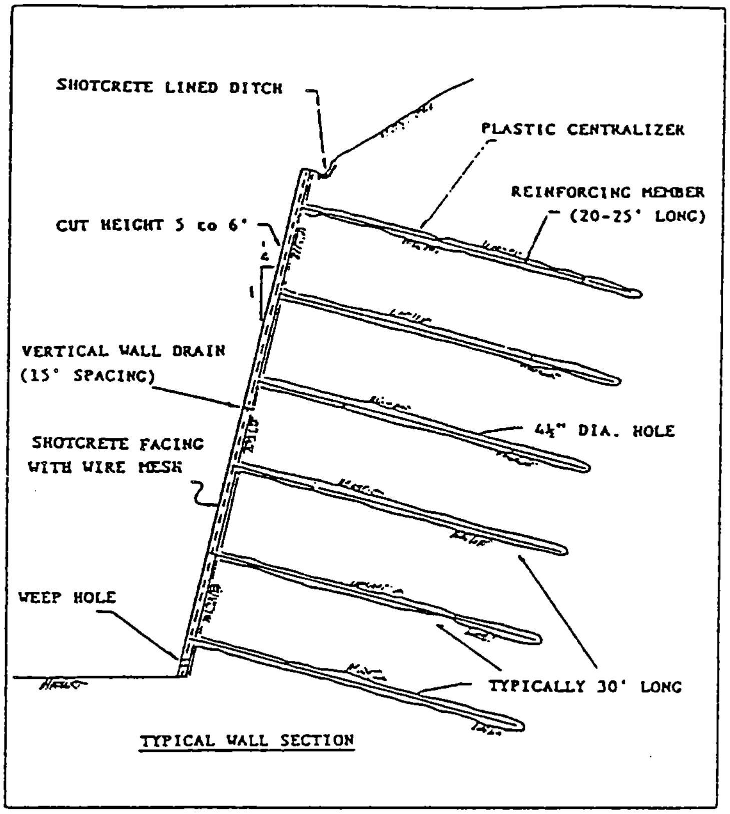

Figure 5. Details of Cumberland Gap Soil Nailing

The slope indicators and the EDM's measurements are in agreement and show a total movement of 1/4 to 3/8 inches at the head of the instrumented nails, which falls within the expected range. The strain gauge data have been more difficult to decipher, showing wide ranges of loadings that could be related to frost conditions. There is also a suspicion that some bending stresses are affecting the bars and, therefore, strain gauges.

This was the first use by the FHWA of nailing to solve a combination of design and environmental problem and was recognized by an award in the "Excellence in Highway Design" 1986 Biennial Awards of the Federal Highway Administration.

SUMMARY AND CONCLUSIONS

Although our present system of contracting for public works is mainly aimed at "build according to plans and specs and by the low bidder", there are still variations that permit innovation. These contract provisions tend to vary from state to state, some are more progressive than others, but almost all have alternative bidding in one form or another. The key here is incentives for the owner, engineer and contractor along with establishing proper basic parameters that keep the process a fair and open one that is not suborned by subjective judgments.

At the Federal level, the Corps of Engineers, the Bureau of Reclamation and the FHWA are all totally committed to Value Engineering in construction to allow for cost savings and innovation. This process has two main drawbacks, schedule constraints that can make it difficult to propose and receive approval of a change in a timely manner and the fact that the savings are shared, sometimes leaving both the owner and the contractor with the feeling that the other got the biggest share of the pie.

As can be seen from the brief examples given in this discussion, our contracting practices are evolving and innovation is moving ahead in retaining wall design and construction. The cost savings and performance of the alternatives presented by contractors have brought the demise of the conventional concrete cantilever closer to a reality each year. With cost savings of 15 to 30% it is worthwhile for the owner and the owner's engineer to take the risk and accept the potential hassle for proposing something that a few people still feel is new and daring.

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T-WALL™ - Engineered for Economy

Thomas C. Neel, P.E.[1]

ABSTRACT: T-WALL is a precast retaining wall system. It is easy to construct and readily available from local precast concrete producers. T-WALL is a gravity-type system which offers technical advantages over cast-in-place concrete walls and mechanically stabilized systems. It can be used for a variety of applications. Various architectural finishes are available. Economy comes from the simplicity of materials, production and construction.

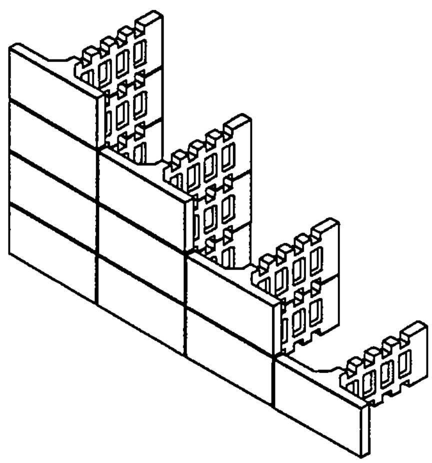

Fig. 1. T-WALL™ RETAINING WALL
(A patented system)

INTRODUCTION

T-WALL is a precast concrete retaining wall system. The system was developed in 1986 and continues the evolution of earth retention systems that has been seen over the last twenty years. T-WALL combines the best features of traditional gravity walls and reinforced soil into a simple economical solution for retaining structures. It has the weight and durability of concrete used in traditional walls, and the economy of frictional resistance used in earth reinforcement systems.

A classification scheme for earth retention structures was proposed by Professor Thomas D. O'Rourke[2] at the recent *Earth Retaining Structure* conference held at Cornell University. Under this scheme, T-WALL would be classified as a hybrid system that combines elements of both internally and externally supported soil.

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1 -President, The Neel Company, Springfield, VA.
2 -Professor, School of Civil and Environmental Engineering, Cornell University, Ithaca, NY.

The standardization of the precast concrete unit has simplified the manufacturing process. Quality in construction has been improved by the ease of installation.

APPLICATIONS

T-WALL has been used to solve a number of site related problems commonly encountered in public works (Fig. 2a, b, c, d, e).

Typical applications include:

- roads
- highways
- site development
- trash transfer stations
- bank stabilization and maintenance
- storm retention basins
- flood control projects
- old retaining wall rehabilitation

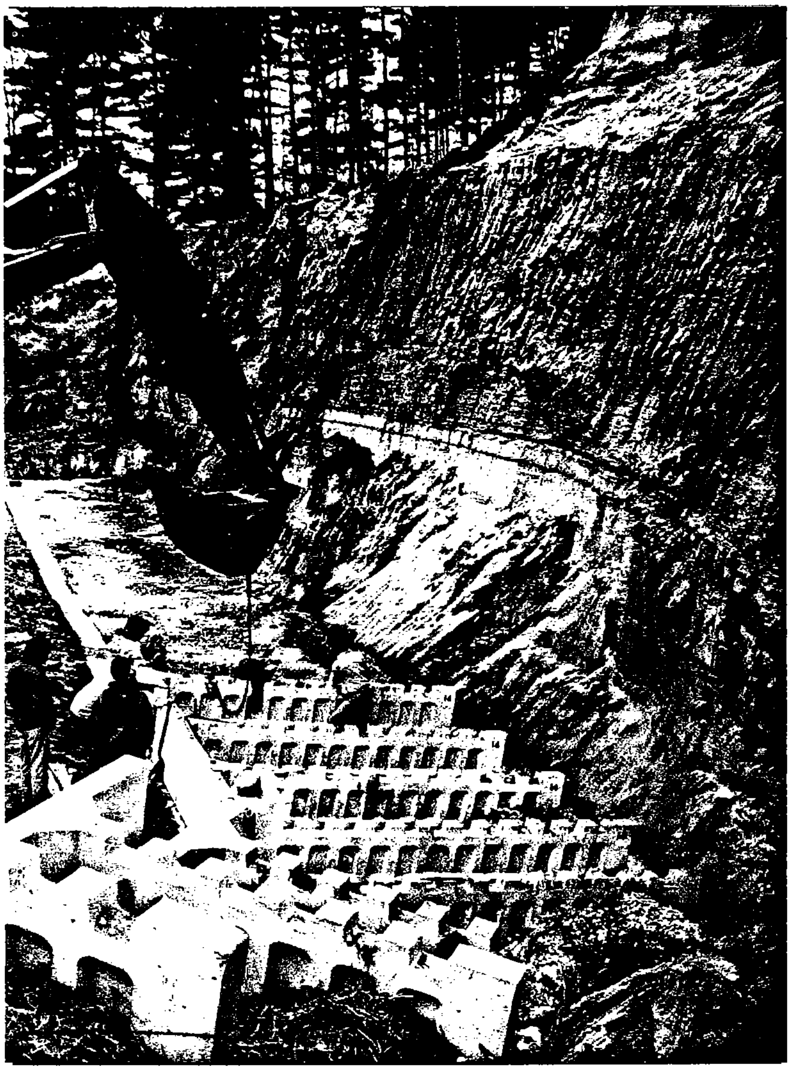


Fig. 2a. Bank Stabilization - Bedford, NH

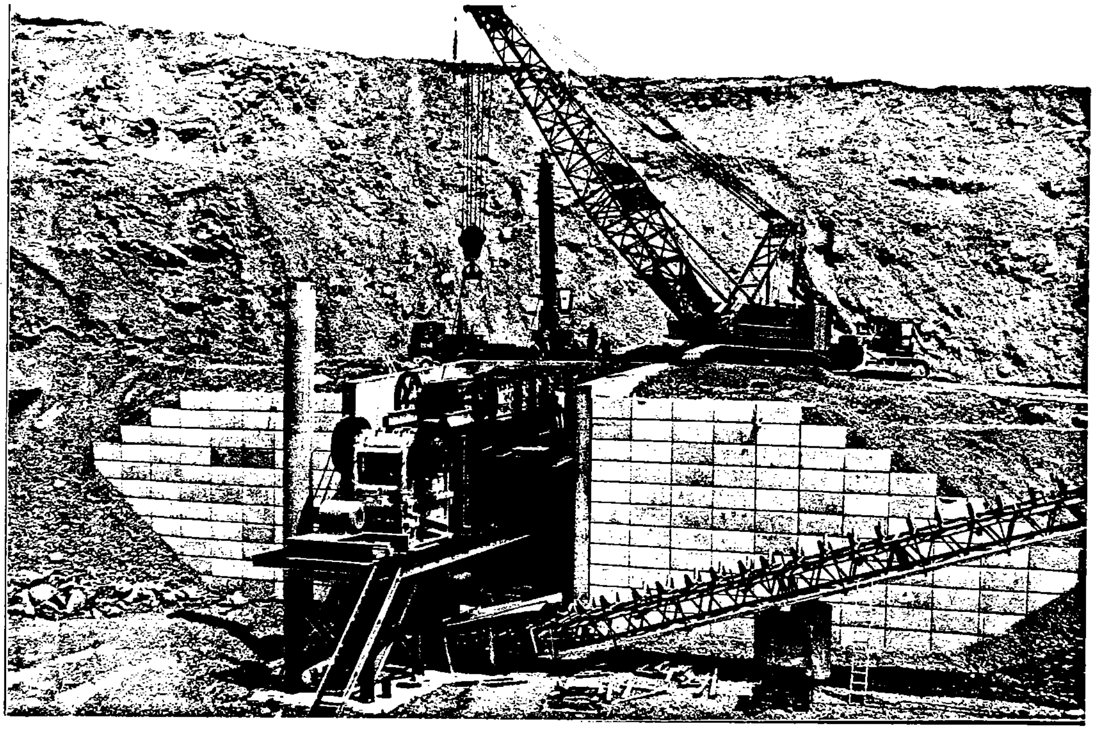


Fig. 2b. New Enterprise Quarry - Roaring Spring, PA

Fig. 2c. Sandstone Road - Vail, CO

APPLICATIONS (con't)

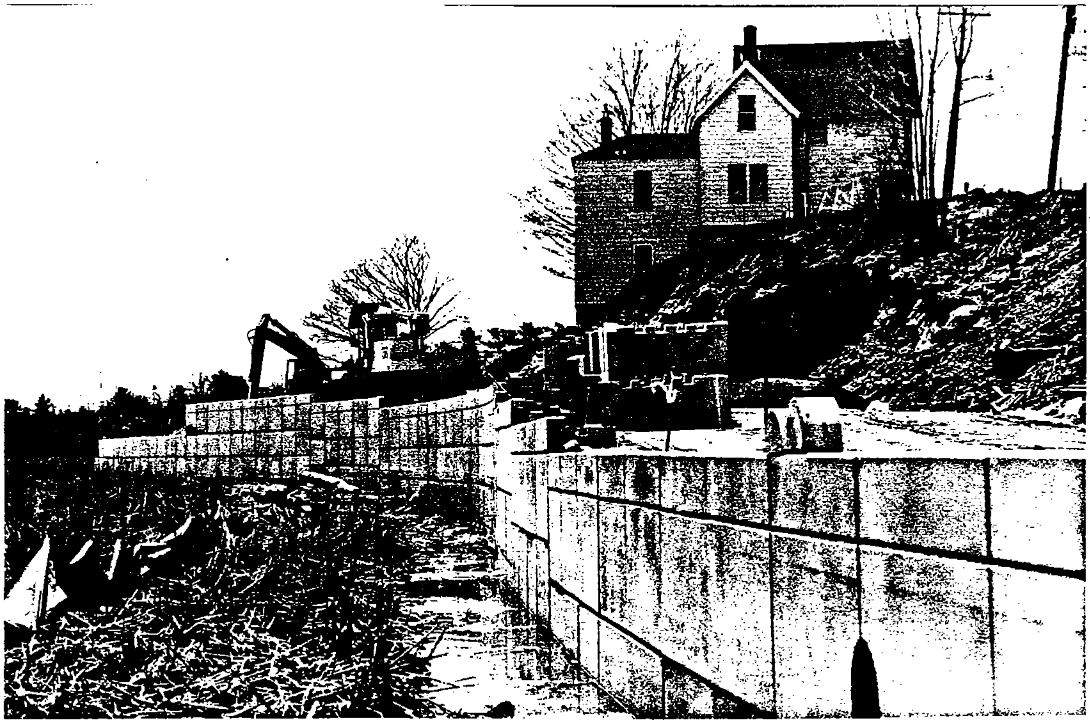


Fig. 2d. Haul Road - Bath, ME

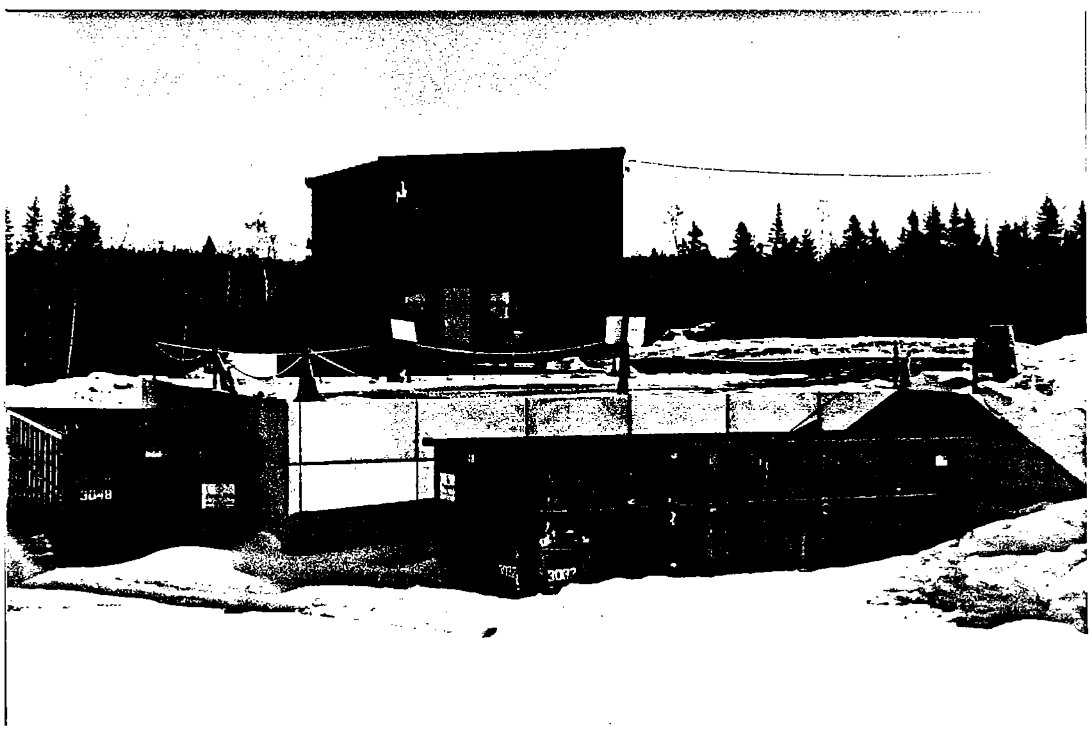
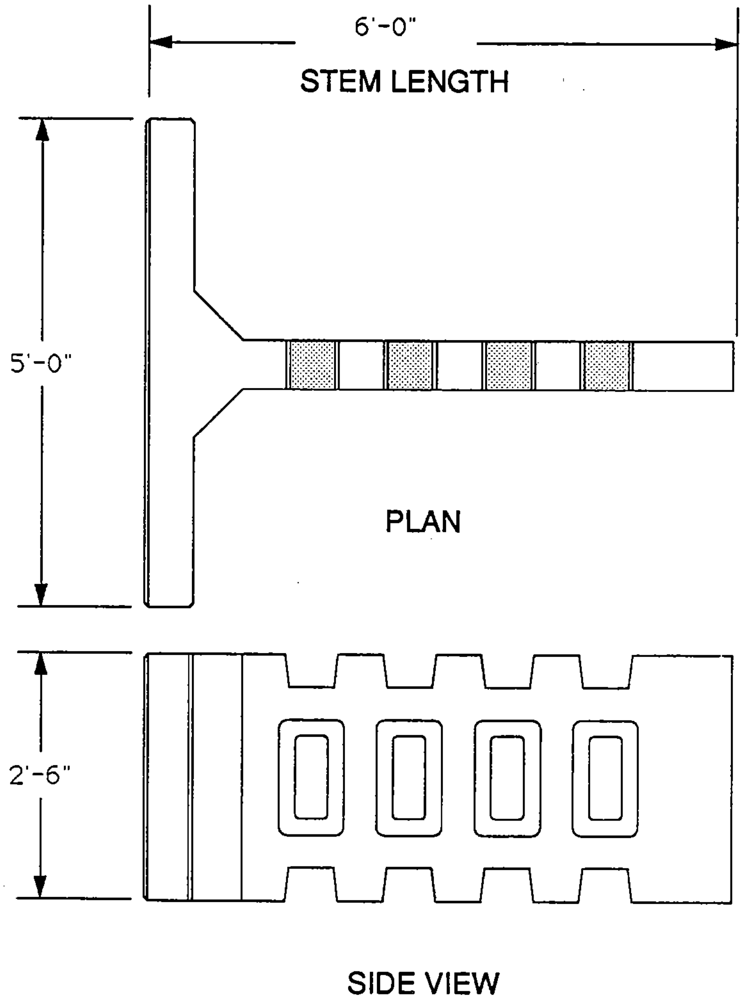


Fig. 2e. Trash Transfer Station - Rangeley, ME

APPLICATIONS (con't)

T-WALL's standard unit size is the key to its versatility. Each unit has a face dimension of 2.5 feet by 5.0 feet and weighs about 2,000 pounds (Fig. 3). The stem lengths vary in 2 foot increments according to height of the wall and the various surcharge loads. Only three standard unit types are needed to construct most retaining walls up to 15 feet high.



STEM LENGTH (FT)	WEIGHT (lbs.)
4	1600
6	1850
8	2100
10	2400

Fig. 3. TYPICAL T-WALL™ UNIT

COMPARISON WITH OTHER RETAINING WALL SYSTEMS

Rigid Structures

T-WALL's joint arrangement makes it a flexible system. This allows it to adjust to the external conditions and imposed loads. For example, the vertical joint arrangement is much less sensitive to overall and/or differential settlement than a rigid structure. This allows a wider range of foundation conditions or may result in economies of foundation treatment.

Crib and Bin Walls

Three features that distinguish T-WALL from most crib wall units are: less pieces to manufacture, handle and install; the ease of compaction between the stems; and the availability of architectural finishes.

Earth Reinforced Systems

The T-WALL stem length is generally 0.6H. Comparable strip or grid reinforcement would be 0.8H. This reduces excavation and backfill for T-WALL. In addition, T-WALL has a minimum base width of 4 feet. The shortest strip or grid is usually 8 feet. Standard T-WALL units can also be built on a batter which further reduces the stem length.

DESIGN

External Stability

Stability calculations are made assuming that the system acts as a rigid body. Lateral earth pressures are computed using the Rankine theory with a vertical pressure plane drawn from the back of the lowest stem.

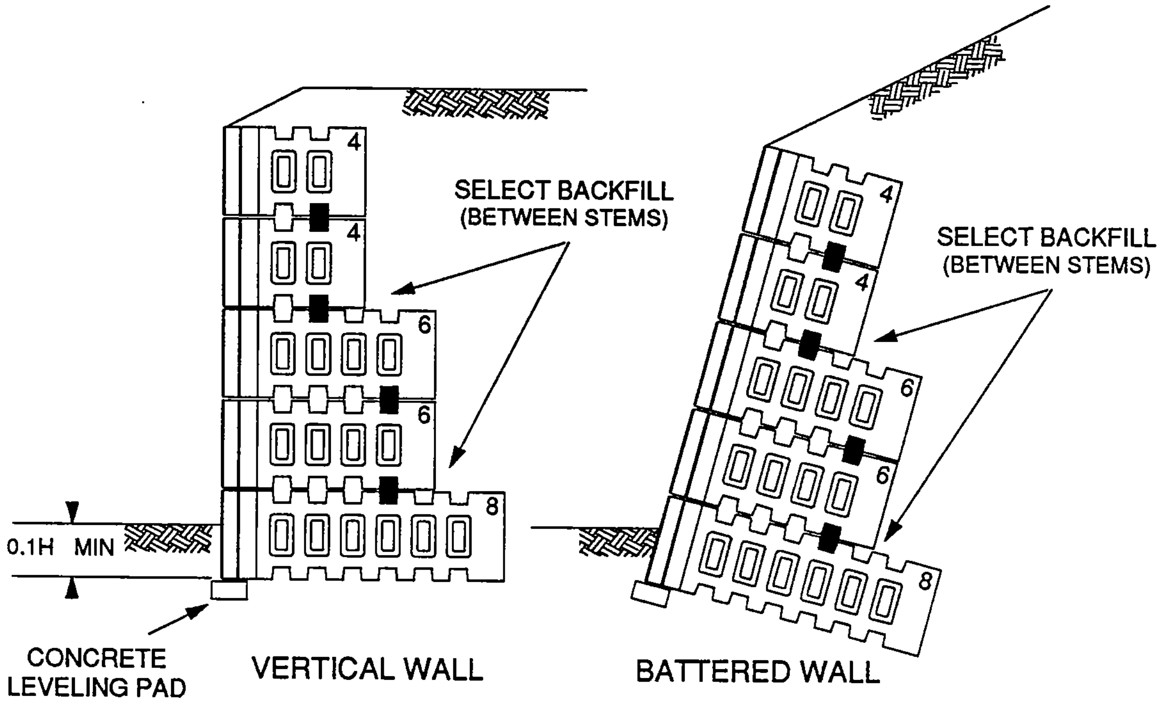
The bearing pressure exerted on a T-WALL foundation is calculated using the Meyerhof distribution which considers an effective footing dimension of the lowest stem length minus two times the eccentricity.

Computations for stability are made at each 2.5 foot level. At each level the stem length is checked to insure that the factors of safety with respect to sliding and overturning are met (Fig. 4).

System Stability

The system stability is a function of the weight of the concrete units, the frictional resistance of the stem, and the vertical connection through the shear keys. T-WALL also benefits from the soil-arching between the stems, but this is not used in the design calculations.

Computations for internal stability are made at each 2.5 foot level by comparing the pullout force generated by the select backfill between the stems to the resisting forces.



GENERAL NOTES:

SELECT BACKFILL BETWEEN STEMS
- 25 % MAX. PASSING #200 SIEVE
- 95 % STANDARD COMPACTION ASTM D-698

UNCLASSIFIED BACKFILL - COMPACTION 90 %

HORIZONTAL JOINT - 1/2" CORK

VERTICAL JOINT - 1/4"± SPACE, FILTER CLOTH BACKING

CONCRETE LEVELING PAD
- 6" x 12"
- 2500 psi CONCRETE
- NO REBAR
- TOLERANCE 1/4" ± IN 10'
- ADD 1/4" FOR EACH JOINT FOR FIELD LAYOUT

CONSTRUCTION - SEE T-WALL CONSTRUCTION PROCEDURES

Fig. 4. TYPICAL T-WALL™ SECTIONS

Select Backfill
Select backfill is required between the stems. Depending on the project's conditions the select backfill may vary in gradation from:

100% passing a 6 inch screen
15% to 30% maximum passing a #200 sieve

For walls up to 20 feet high under normal conditions select backfill with fines up to 30% maximum have performed well.

The select backfill is varied for specific site conditions. For example, where the wall was constructed in a tidal zone, the select backfill specification limited the fines to a maximum of 15% to insure good drainage and prevent a hydrostatic differential.

Drainage
The vertical joint system provides a full height weep hole every 5 feet. Due to the excellent drainage, the select backfill does not require a high degree of permeability.

MANUFACTURING

The units are precast under plant controlled conditions which provide good quality control. The units are cast in steel forms. A 4,000 psi concrete strength is required. The mix design is specified by the local producer or owner. The rebar is designed in accordance with AASHTO and ACI codes.

The architectural finish is a function of the owner's imagination and pocketbook. The finish may be obtained by using a variety of form liners, coatings, exposed aggregate or sand blasting.

TECHNICAL ASSISTANCE

Specific project engineering is provided by consulting engineers or the local producer. The local producers provide construction assistance to the contractor. The local producer works in partnership with The Neel Company which provides a central source of information and experience. This insures product uniformity and an exchange of information on a national basis. The *T-WALL Design Guide & Technical Information* booklet and the *T-WALL Construction Procedures* manual are examples of the technical assistance.

CONSTRUCTION

Installation

T-WALL construction is fast and easy (Fig. 5a, b). It begins with a cast-in-place leveling pad, 6" deep by 12" wide. The leveling pad grade is checked and the wall line is marked. The existing ground is proof rolled to insure a stable foundation. The units, which weigh about 2,000 pounds each, are placed on the leveling pad. The bottom course is leveled and backfilled.

The select backfill is compacted with a walk-behind vibratory or a drum roller to 95% standard proctor density. Backfilling and compacting is facilitated by the open back.

A concrete shear key is placed in the notch of the stem to prevent the new unit from moving while the select backfill is being placed and compacted. The horizontal joint material is a cork strip 4" wide. The vertical joint material is a 12" strip of geotextile cloth.

The next course of units is placed and the cycle repeated. T-WALL is constructed in 2.5 foot lifts.

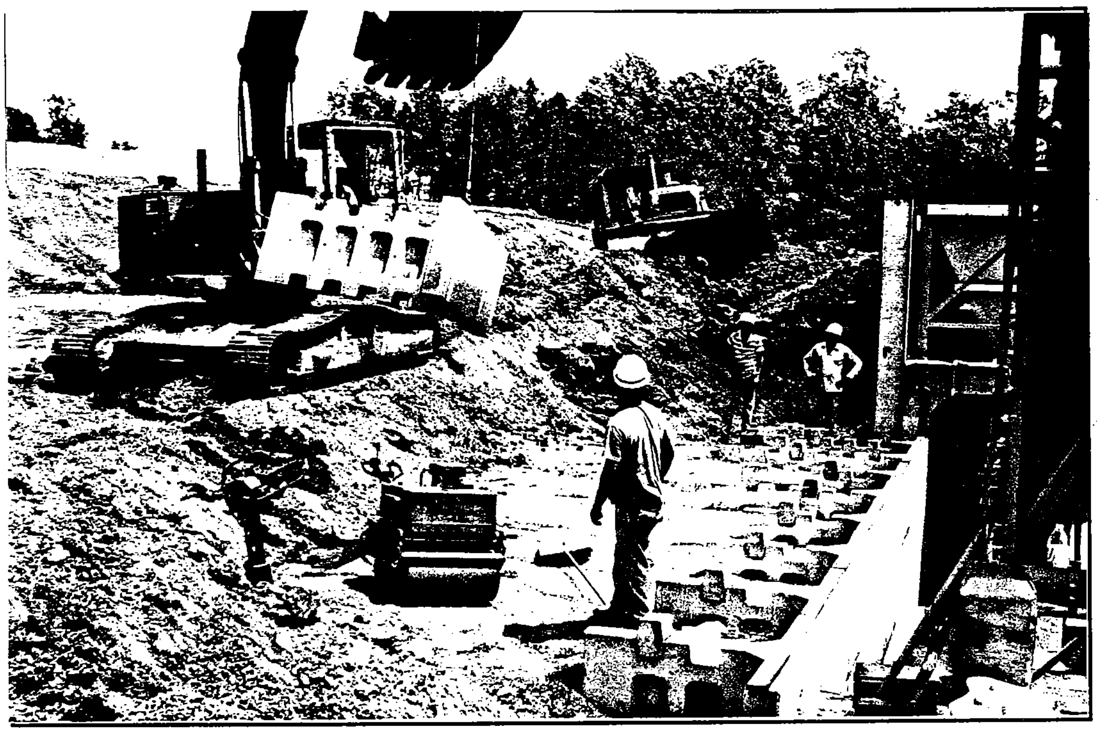
ADMINISTRATION

T-WALL is readily available from local precast concrete producers all over the country. In fact, for rush projects, the production capability and inventory of several producers has been mobilized for a quick start.

Contractors who can set other precast items such as drop inlets and manholes can construct T-WALL. This allows many small contractors to compete for the work. And since the site contractor can build the wall, the number of contractors on the site may be reduced.

Project schedules are improved by: the ease of construction, the ability to work in cold or wet weather, and the ability to produce units and prepare the site concurrently.

Fig. 5a, b. Asphalt Plant - Herndon VA



CONSTRUCTION

COST

The cost of the wall for most owners includes: all grading necessary for wall construction, compaction of the wall foundation, general and local dewatering, construction of the leveling pad, erection of the precast concrete units, and placement and compaction of the backfill between the stems (Fig. 6).

T-WALL In-Place Cost Estimate
12' high retaining structure

Item	$/sq. ft.
Leveling Pad (@$8.00/lf)	$0.70
Material, FOB jobsite	12.50
Installation 400 sf/day 3 man crew 1 crane (12 ton) operated Contractor's markup	5.00
Backfill (@$10.00/cy) 25% max passing #200 sieve	2.20
Total In-place	$20.40/sf

Fig. 6. EXAMPLE OF COST COMPONENTS

INNOVATION

Over 100 T-WALL projects have been built in the last three years. It has been used successfully in the very cost competitive industrial market. However, it has been very difficult to find highway departments that will establish a positive plan of engineering and field evaluation to investigate the advantages offered by the system.

Highway departments have the resources and opportunities to foster innovation for which they will receive the benefits. This simple system based on gravity and friction has been proven in the commercial market. Its many advantages will also work on highway projects.

ACKNOWLEDGEMENTS

Many people have contributed to the development and success of T-WALL. These people include the inventor, those who developed the engineering and those who helped in the development of the manufacturing and construction techniques.

Individually I would like to thank Ray O'Neill who had the original concept and Bruce Hanna who did the initial engineering work and who understood that the system combined the good features of several older systems.

I would like to give special thanks to the owners who took the time and had the courage to try a new product.

SLOPE FAILURE PROBABILITY FOR LAYERED SOILS

Steven R. Garrett and Sam I. Thornton

ABSTRACT

This paper describes a statistical method to evaluate the stability of an earth slope. The point estimate method allows computation of a "probability of failure" in slopes containing layered soils which contain either cohesion or internal friction.

The method can be summarized by the following steps:

1. Find the mean and standard deviation of the values for cohesion and internal friction.
2. Find the high and low values for cohesion and internal friction for each layer.
3. Find the factors of safety for the slope using the combinations of maximum and minimum values of cohesion and internal friction.
4. Find the expected value and standard deviation of the factor of safety.
5. Find the "probability of failure" from a normal distribution table.

The method is a useful engineering tool to design safer more economic slopes by accounting for the uniformity of a soil deposit and extent of soil testing.

INTRODUCTION

In order to check the stability of a highway slope, a "factor of safety" (FS) is traditionally calculated. Highway personnel, however, are most interested in whether the slope will be stable or fail. The probability of failure is a better measure of stability than a comparison of soil stresses and strength.

An example of the probability of failure was presented at the 1988 Highway Geology Symposium by J. R. Verduin and C. W. Lovell. In the example, the expected FS was 1.413. However, the probability that the FS was below 1.0 where failure is assumed to occur is 2.87%.

High probabilities of failure can occur when the strength of the soil is variable or based on only a few tests. Soil variability can be compensated for by using "Engineering Judgement" and increasing the required FS. Knowing the probability of failure improves judgement because it provides a rational basis for making a safe and economical design for highway slopes.

The Verduin and Lovell paper described the probability of failure method for one soil with cohesion and internal friction. This paper describes the method for multiple layers of soil, each soil having only cohesion or internal friction.

BACKGROUND

The probability of failure method is based on the "Point Estimate Method" (PEM) which was developed by Rosenblueth (1975 and 1981) and described by Harr (1987). In the PEM, a distribution of the variable must be found or assumed. If a normal distribution is assumed (not unreasonable because all that is known at the start is the mean and standard deviation of the soil strength), the problem is simplified. Details of the PEM development and a discussion of other distributions are contained in a thesis by Garrett (1989).

The basic equations necessary to solve a slope stability problem by finding the probability of failure are given below.

Basic Statistics:

the mean: $$\bar{X} = \frac{X_1+X_2+X_3+.....+X_n}{n}$$ equation 1

where: $X_1, X_2, X_3, ... X_n$ are values in a set of data (strength tests)

n is the total number of values

the variance: $$\sigma^2 = \frac{\Sigma(Xi-\bar{X})^2}{n-1}$$ equation 2

where: Xi are individual values in the data set

The variance is a measure of the scatter of a random variable about its mean.

the standard deviation: $$\sigma = \left[\frac{\Sigma(Xi-\bar{X})^2}{(n-1)}\right]^{.5}$$ equation 3

The standard deviation (σ) is a more useful measure of the dispersion of a random variable about its mean.

Point Estimate Method For Slopes:

Values for strength a standard deviation away from the mean are called strength plus (X+) and strength minus (X-) values.

strength plus = $X+ = \bar{X} + \sigma(x)$ equation 4

where $\bar{X}$ is the mean strength value (either cohesion or internal friction)
$\sigma(x)$ is the standard deviation of the strength in that layer

strength minus = $X- = \bar{X} - \sigma(x)$ equation 5

For layered soils, a FS must be found for each combination of soil strength x+ and x-. The number of FS to be found then is 2^n, where n is the number of soil layers. A soil with two layers of soil would require four FS's. Three soil layers would require eight FS's and four soil layers sixteen.

The symbol FS++ is used for the FS for a two layer slope with x+ used for the strength in both layers. FS+- is used for the FS with x+ in the first layer and x- in the second layer. If the slope has three layers, FS+++ is used for the FS using x+ values in all three layers.

Normal Distribution:

The expected FS, E(FS), is the mean of the factor of safety.

For two layers: $E(FS) = 1/4\ (FS{++} + FS{+-} + FS{-+} + FS{--})$ equation 6

The expected FS squared, $E(FS)^2$, is the square of the value from equation 6.

$$E(FS)^2 = [E(FS)]^2 \qquad \text{equation 7}$$

The expected value of the squared FS, $E(FS^2)$, is the mean of the squared SF values.

For two layers: $E(FS^2) = 1/4\ [(FS{++})^2+(FS{+-})^2+(FS{-+})^2+(FS{--})^2]$ equation 8

The standard deviation of the FS's then is:

$$\sigma(FS) = [E(FS^2) - E(FS)^2]^{.5} \qquad \text{equation 9}$$

The standardization variable, Z, is

$$Z = \frac{FS-E(FS}{\sigma(FS)} \qquad \text{equation 10}$$

where FS is the cutoff value of the normal distribution table (e.g. FS = 1.0)

From the standardized variable, Z, a normal distribution table (Table 1) can be used to find the probability that a value will be less than Z.

Table 1
The Normal Distribution Table

Z	0	-.5	-1.0	-1.5	-2.0	-2.5	-3.0
Probability	.5	.3085	.1587	.0668	.0228	.0062	.0013

EXAMPLE

Application of the slope failure probability method for layered soils is best shown by an example (Figure 1).

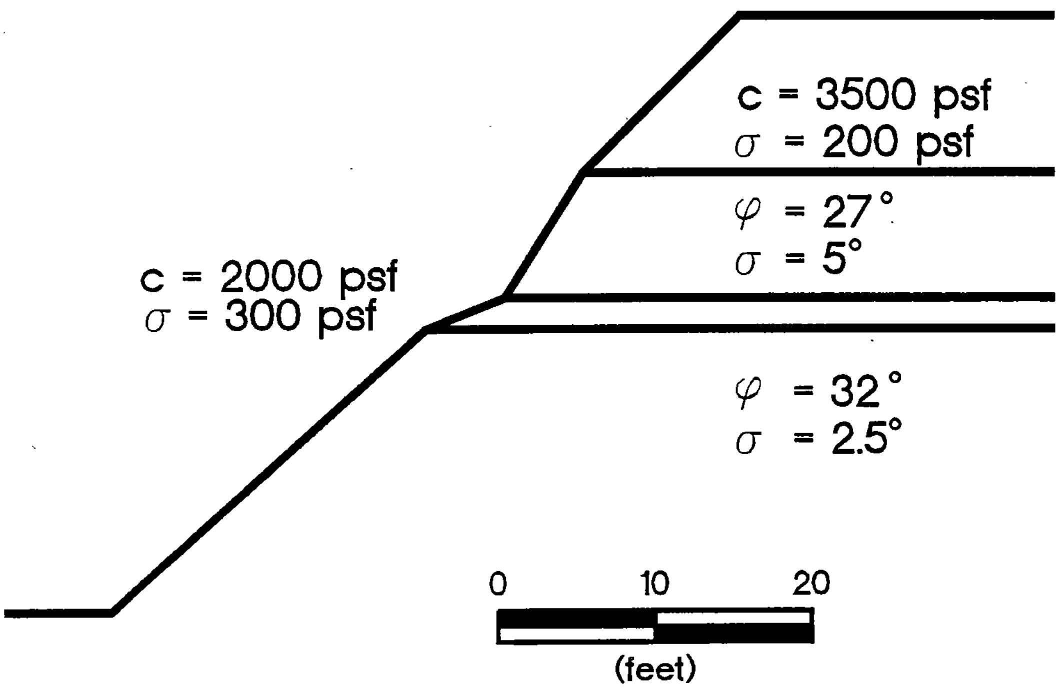


Figure 1
Example Slope

High and low values for soil strength obtained from equations 4 and 5 are contained in Table 2.

Table 2
High and Low Strength Values

Soil Layer	Soil Strength +	Soil Strength -
1	3700 psf	3300 psf
2	32°	22°
3	2300 psf	1700 psf
4	34.5°	29.5°

The FS for the slope in Figure 1 are then found by the Bishop Method for all combinations of soil strength (Table 3).

Table 3
FS's for High and Low Strength Combinations

Strength Combination	FS
++++	1.4024
+++-	1.1966
++--	1.1428
+---	1.1239
----	1.1235
-+--	1.1424
-++-	1.1966
-+++	1.4021
--+-	1.1798
--++	1.3786
---+	1.3126
-+-+	1.3352
+-+-	1.1798
+--+	1.3130
+-++	1.3790
++-+	1.3356

The expected value of the FS, E(FS), is found from Equation 6.

$$E[FS] = 1/16\ [1.4024 + 1.1966 + 1.1428 + 1.1239 + 1.1235 + 1.1424 + 1.1966 + 1.4021 + 1.1798 + 1.3786 + 1.3126 + 1.3352 + 1.1798 + 1.3130 + 1.3790 + 1.3356] = 1.2590$$

Next, the expected value of the squared FS's is found from equation 8.

$$E[FS^2] = 1/16\ [1.4024^2 + 1.1966^2 + 1.1428^2 + 1.1239^2 + 1.1235^2 + 1.1424^2 + 1.1966^2 + 1.4021^2 + 1.1798^2 + 1.3786^2 + 1.3126^2 + 1.3352^2 + 1.1798^2 + 1.3130^2 + 1.3790^2 + 1.3356^2] = 1.5958$$

The standard deviation of the FS's from equation 9 is 0.1035.

$$\sigma[FS] = (1.5958 - 1.2590^2)^{.5}$$

Finally, the probability that the FS is less than 1.0 is found from equation 10 and a normal distribution table (Table 1).

$$z = \frac{1.00 - 1.2590}{0.1035} = 2.50$$

The probability that the FS is less than one is 0.62%.

EXAMPLE COMPARISON

The slope in the example by Verduin and Lovell had a FS of 1.413 but the probability of failure was 2.87%. The four layer example presented in this paper had a FS of only 1.259 but it's probability of failure was only 0.62%. The slope in the Verduin and Lovell example then is over four times as likely to fail.

CONCLUSION

The slope failure probability method provides useful information to design safe economical highway slopes.

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FEDERAL HIGHWAY ADMINISTRATION'S TECHNOLOGY TRANSFER ACTIVITIES IN GEOTECHNICAL ENGINEERING

Chien-Tan Chang
Highway Engineer
Office of Implementation
Federal Highway Administration (FHWA)
McLean, Virginia

Background

The cost of highway construction has increased rapidly over the last 20 years. The high cost of right-of-way forces us to construct highways over the worst soil conditions. Choice of soil and rock for borrow and base materials are becoming scarce, requiring the use of marginal construction material.

The need to apply the principle of modern geotechnical engineering in the location, design, and construction of today's highway program is increasing. Bold new design techniques make an accurate description and prediction of soil behavior necessary if costly failure or over-design are to be avoided. Minimization of disturbance to the environment are needed to assure that best use is material requiring a design tailored to the actual in place condition.

Geotechnical expertise is unique in solving many of today's problems in highway construction. Knowledge of soil and rock behavior minimizes adverse environmental effects by minimizing erosion, slope stability problems, and making best use of available material.

Research in geotechnical engineering has resulted in many new concepts and new methods for highway design and construction, which if utilized, will reduce construction and maintenance cost considerably. However, most research output for geotechnical engineering is technical. Most of the research output requires additional development and specific implementation effort before utilized by practicing engineers.

FHWA's Technology Transfer Programs

The Technology Transfer (T^2) programs of the FHWA has been in operation for about 20 years. Presently FHWA has more than 100 T^2 activities in highway engineering disciplines. Many of these activities are related to geotechnical engineering.

The implementation of new technologies in geotechnical engineering has been conducted using a variety of techniques. These methods include the development and presentation of training courses, demonstration projects, presentation of workshops, equipment development, field evaluation of new materials and techniques, and preparation of technology sharing reports, implementation packages and computer programs.

New, cost-effective technology is promoted at the Headquarters level of the FHWA primarily by the following three groups:

- The Office of Implementation translates research findings into products that can be directly implemented by the engineering community, such as users manuals, tutorials, videotapes, handbooks, and brochures.

- The Demonstration Projects Division promotes new research results and innovative engineering practices through hands-on demonstrations, workshops, and the evaluation of experimental features used on Federal-aid projects. Technical and limited financial assistance are available to evaluate and incorporate new technologies in the field.

- The National Highway Institute (NHI) develops and presents training courses to the States, and local governments. In addition, it oversees the Rural Technical Assistance Program (RTAP), which includes a network of technology transfer centers that provide technical assistance for rural agencies. In addition to the Headquarters level technology transfer offices, T^2 information is available from FHWA personnel at the Division and Regional levels of the agency. The FHWA maintains Division offices throughout the United States, and with the exceptions of Ames, Iowa, and Baltimore, Maryland, these offices are located in State capitals. Offices are also maintained in the District of Columbia and in Puerto Rico.

All inquiries about available information should be directed first to the Division offices. If additional assistance is required, the Regional staff and Headquarters staff are available. Information is also available for local agencies from the NHI sponsored RTAP Centers. These centers are located in most States, at colleges, universities, or State Department of Transportation.

The following is a summary of major technology transfer (including research) activities related to rock and soil slopes, and soil improvements:

1. Stone Columns

Stone columns have been used since the 1950's as a technique for improving both cohesive soils and silty sands. Potential applications include (1) stabilizing foundation soils, (2) supporting structures, (3) landslides stabilizing and (4) reducing liquefaction potential of clean sands. The high potential for beneficial use in highway applications prompted a comprehensive investigation to determine how and why the system works so well and to develop appropriate design and construction guidelines. The FHWA guidelines report (FHWA/RD-83-026) describes construction , field inspection, and design aspects of stone columns. Also, several case histories are described. Bearing capacity, settlement and stability design examples are given in the appendices.

2. Prefabricated Drainage Systems

One of the most cost-effective methods of soft ground improvement is drainage. The creation of artificial drainage paths to shorten the distance to a drainage boundary provides engineering control and construction expediency which improves structural performance and reduces maintenance costs. The use of prefabricated drainage systems has rapidly increased during the last 10 years because they are cheaper, easier to install, and environmentally less disruptive than sand or stone-drainage systems.

Although they have many significant advantages over conventional drains, there were many unanswered questions regarding design assumption, laboratory screening tests, quality control, installation methods, and long-term durability. A comprehensive investigation of prefabricated vertical drains (wicks) and geocomposite drainage boards was undertaken to develop rational engineering guidelines and recommended generic specifications. The results from the wick drain research can be found in FHWA-Rd-86-168, and the geocomposite guidelines that are in FHWA/RD-86-171.

3. Dynamic Compaction

Compaction is also a very cost-effective soil improvement technique. Dynamic compaction is the densification of soil deposits by means of repeatedly dropping a heavy weight onto the ground surface. Although simple in concept, there were many unanswered questions concerning the design mechanism and construction control specifications. A research study was initiated to evaluate the method and to develop design and construction guidelines for using dynamic compaction procedures for improving subsurface ground conditions beneath highway embankments and structures. A series of field experiments were conducted to investigate soil and tamping parameters involved in the dynamic compaction process. Instrumentation was installed to monitor ground vibrations, horizontal and vertical displacements, pore pressures, acceleration and speed of the tamping weight, penetration of the weight into the ground, and degree of improvement achieved. The results of the experimental tests were used to develop generic specifications and a technical guidelines manual (FHWA/RD-86-133).

4. Stability of Rock Walls and Fills

A Coordinated Federal Land Technology Implementation Program (CTIP) research study to develop a computer program to analyze slope stability of massive rock structures was recently completed. The computer program and corresponding users manual will be distributed in 1990 or early 1991.

5. Rockfall Hazard Rating System

A pooled-fund Highway Planning and Research Program (HP&R) study was initiated to develop a rational procedure for rating and prioritizing hazardous rockfall situations in mountainous terrain. A series of one day courses is being conducted.

6. Permanent Ground Anchors

A contract research study was recently initiated to evaluate and improve existing design and construction guidelines for permanent ground anchors that were developed and introduced under Demonstration Project 68. Model and full-scale tests will be conducted to determine rational allowable loads for permanent ground anchored walls. Measurements will be made on the anchors, backfill soil, soldier piles, and on the back of the wall to determine soil/anchor interaction, earth pressure on the wall, grouting requirements, bond zone load transfer, angle of wall friction, and soldier pile load transfer. The results will be used to modify existing specifications and design guides.

7. Pressuremeter Manual and Video Tape

An implementation package on the pressuremeter has been developed for distribution in late 1989. In addition to a users manual, a videotape describing various types of pressuremeters, their application in highway engineering and step-by-step soil testing procedures have been developed to promote increased use of this device.

8. Geosynthetic Materials

A technical summary of geotextile specifications has been prepared and recently distributed to assist State highway agencies in improving their specifications. The FHWA is participating with 20 other government and private sector organizations in a consortium to support a research institute for Geosynthetic Materials. The Geosynthetics Research Institute (GRI) was organized by Drexel University to engage in short- and long-range research associated with the use of geosynethetics, including durability, degradation, clogging and aging.

9. Behavior of Reinforced Soil

A series of laboratory model tests and full-scale field tests were recently conducted on several types of soil reinforcement systems. The experimental test data was used to evaluate soil and reinforcement parameters required for verification and refinement of existing design and construction guidelines. An engineering manual was written in generic terms and should be available by the end of 1990.

10. Corrosion/Durability of Soil Reinforcing Elements

A research study is currently underway to develop new techniques and equipment to determine corrosion potential of metallic reinforcing elements and durability of non-metallic elements as relegated to the design life of the reinforced soil structures. The contractor will also develop recommendations for laboratory tests for the analysis of backfill materials and techniques for corrosion monitoring and inspection of existing reinforced soil structures. The study report should be available by the end of 1990.

11. Soil Nailing for Highway Cutslopes

A series of laboratory model tests and full-scale field tests were recently completed to evaluate soil and reinforcement parameters that are involved in the design and construction procedures for soil-nailing. Data analysis and evaluation of existing procedures was used to develop a comprehensive design and construction guidelines manual for using soil-nailing techniques to stabilize soil cutslopes. The manual should be available in December 1990.

12. Microcomputer Programs in Geotechnical Applications

The microcomputer industry has undergone rapid changes in recent years. New developments in hardware and software make the use of the microcomputer in civil engineering applications more feasible, practical, and almost necessary.

One definite advantage in the use of microcomputers is their ability to perform repetitive calculations quickly, conveniently, and efficiently. The microcomputer can be used to solve many geotechnical problems needing repetitive and yet complicated calculations, such as analyzing embankment and foundation deformations, estimating pile behavior under static and dynamic forces, and calculating foundation settlements. Attached is a list of geotechnical microcomputer programs available at FHWA's Office of Implementation.

In addition to the above activities, FHWA has been involved in the developing and conducting of training courses. Many of these courses were accompanied by comprehensive design and construction manual in the related subjects:

1. Advanced Slope Stability
2. Slope Maintenance and Slide Restoration
3. Geotextile Engineering
4. Rock Blasting
5. Rock Slope Engineering
7. Seismic Design for Foundations
8. Soil Stabilization

Summary

A significant amount of current and future research and technology transfer activities will be carried out to accomplish the overall objective of developing improved foundation engineering, slope engineering and ground improvement techniques. Inputs from the Transportation Research Board, State highway agencies, universities and the rest of the worldwide geotechnical community will be used to strengthen the proposed technical approach. Results from the previous research and technology transfer efforts will be combined with the expected results from the T^2 effort to accomplish the overall objective.

FHWA Office of Implementation
List of Geotechnical Microcomputer Programs

STABL4M:

Microcomputer program for the general solution of two dimensional slope stability problems using simplified Janbu's and Bishop's methods.

STABL5M:*

Microcomputer program for the general solution of two dimensional slope stability problems using simplified Janbu's, Bishop's methods and Spencer's method.

STABL6:*

Simplified Bishop's method of analysis in STABL microcomputer program was modified to analyze reinforced embankments.

System requirements:
IBM-XT, AT or compatible
MS-DOS 2.1 or higher
256K RAM
Optional: Printer, Graphics display, Math Co-Processor, HP-Plotter.

COM624P:

Microcomputer program to analyze the behavior of piles or drilled shafts, subjected to lateral loads using P-Y curve method.

System requirements:
IBM-XT, AT or compatible
MS-DOS 2.1 or higher
256K RAM
Optional: Printer, Graphics display, Math Co-Processor, HP-Plotter.

WEAP87:

Microcomputer program that simulates a foundation pile under the action of an impact pile driving hammer, using Wave Equation Analysis.

System requirements:
IBM-XT, AT or compatible
MS-DOS 2.1 or higher
320K RAM
Optional: Printer, Graphics display, Math Co-Processor.

* These programs are not distributed by FHWA.

EMBANK:

Microcomputer program to determine one dimensional compression settlement due to embankment loads.

System requirements:
- IBM-XT, AT or compatible
- MS-DOS 2.1 or higher
- 256K RAM

Optional: Printer

CBEAR:

Microcomputer program for bearing capacity analysis of shallow foundations.

System requirements:
- IBM-XT, AT or compatible
- MS-DOS 2.1 or higher
- 256K RAM

Optional: Printer

SPILE:

Microcomputer program for determining the ultimate static pile capacity in cohesive and cohesionless soils.

System requirements:
- IBM-XT, AT or compatible
- MS-DOS 2.1 or higher
- 256K RAM

Optional: Printer

ESAC:**

Microcomputer program for studying the quasi-static geomechanical behavior of engineered surface facilities such as slopes and bridge abutments constructed primarily using rock and/or rock fill material.

System requirements:
- IBM-XT, AT or compatible
- IBM-compatible CGA, EGA or VGA monitor
- MS-DOS 2.1 or higher
- 640K RAM and 10 Mb or larger hard disk

Optional: Printer, HP-Plotter

** This program was developed by FHWA Office of R&D.

Additional geotechnical microcomputer programs:

1. Machanically Stabilized Earth Walls
   ----- Under development

2. Reinforcement Slopes and Reinforcement of Embankments on Soft Foundations
   ----- Under development

3. Soil Nailing
   ----- To be developed

DATA ACQUISITION SYSTEM

FOR

MECHANICAL DUTCH CONE PENETROMETER

Prepared for Proceedings of 41st Annual

Highway Geology Symposium

by

Steven M. Huddleson

Chief, Geology and Foundation Exploration
New Mexico State Highway and
Transportation Department

Santa Fe, New Mexico

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ABSTRACT

The purchase of a Compac Portable III computer in 1987 created the opportunity to develop uses for micro computers in field applications. One of the first projects undertaken was an attempt to digitize the output from a Hogentogler Mechanical Dutch Cone device and input the resulting data into the Compac computer. Under a research grant from the New Mexico State Highway and Transportation Department, graduate students from the University of New Mexico installed an in-line pressure transducer and an analog to digital converter on an expansion chassis for the Compac Portable III computer. A driver was written in BASIC to control the data acquisition. Field tests showed that the algorithm used to determine maximum cone and frictions values was not valid. An updated version of the driver was developed and testing took place on Interstate 25, north of Albuquerque. This new driver shows promise and further development is underway.

SOFTWARE

Originally purchased in 1978, the Hogentogler Dutch Cone Penetrometer saw limited use in the department, and serious doubts existed as to the reliability of the data. After many years of non-use, the equipment was calibrated and put back into service. A Lotus 123 spreadsheet was developed to reduce and graph the data. Formulas in the spreadsheet convert gauge readings from Kilo-Newtons to pounds force. Cone resistance is subtracted from the Sleeve plus Cone reading to determine the Sleeve Resistance. The weight of the inner rods, depending on the depth of the test, is determined through modular arithmetic and added to the pressure reading. Although this value is nominal in shallow tests, its inclusion in the total force was easily accomplished and further increases the accuracy of the reading. The cone resistance, sleeve resistance, and friction ratio can then be graphed in a very presentable format using the Lotus Printgraph facility.

The spreadsheet contains several macro's (small internal programs) to facilitate data input, and others are being created to further facilitate graphing. Later versions of this Lotus spreadsheet allow a comparison of the data obtained from gauge readings and the data from the pressure transducer. This method of obtaining data reduction through Lotus has several advantages. With minimal training, a very presentable graphic representation can be obtained for reports and project files. Transducer values generated in the field can be imported directly into the Lotus spreadsheet, eliminating the possibility of data input errors, although at this time all data is input manually using the built-in macro. The graphic output can also be enhanced through other commercial software packages to create a more professional output. The Lotus spreadsheet can also be menu driven through additional macro commands, allowing operators who are not entirely familiar with Lotus to prepare data.

Data acquisition requires a driver program to convert the analog electrical signal to digital data capable of being interpreted by the computer. This program was written in Quick Basic version 4.5. Initially the program used an algorithm which assumed that the pressure curve obtained in a test would exhibit a plateau as the maximum tip extension and sleeve extension occurred. This approach did not reckon with the extreme sensitivity of the data signal. During actual field testing, it was observed that any time the pressure stabilized the program picked a data value. This approach was obviously incorrect and the program was scrapped. A second version was written which was more efficient in terms of computer memory and running time with increased readability. At this point it was assumed that the transducer and associated hardware was reliable and accurate, and the only problem was in picking the pressure value at the time of cone and sleeve extension.

In the second version values for cone and sleeve resistance are selected by the computer operator by pressing the computer's return key at the moment the 'J' bolt indicates maximum extension has occurred. In use, the program is in an idle state until the threshold pressure is reached. As that pressure is detected a looping mode begins, and pressure values are stored in RAM. This looping time is determined by the operator according to conditions. Up to 250 data points can be stored for each test, allowing a sufficient length of time for the operator of the dutch cone to push at a steady 4 feet per minute rate. As the computer operator observes the test, he presses the return key at the moment the 'J' bolt indicates that the cone has reached its maximum extension. The program stores the pressure value at that instant corresponding to the Cone Resistance value. As the 'J' bolt continues to move, the operator presses the return key again to store the Sleeve plus Cone resistance. This procedure can be repeated up to five times in case the operator feels that he has picked an incorrect value. The 250 continuous pressure values are stored on the hard disk upon completion of the push along with the instantaneous pressure values corresponding to cone and sleeve loads. The entire process can then be repeated after advancing the rods for the next depth. Typical pressure curves are shown in Figures 1 through 3.

After each test location is completed, the data should then be copied from the hard disk to a high density floppy disk for storage. The hard disk can then be erased. Because of the disk access time, it is necessary to initially store this data to the hard disk and transfer it later. The DCP driver program stores the data in files labeled with the date and location of the test and can be called upon to print summaries or view the graphic representation of each pressure curve.

Software problems have remained the most challenging of the DCP conversion. Initial comparison of values obtained from gauge readings and from transducer readings have shown that the Cone Resistance values compare quite nicely with very few disparate values. A plot of some typical test holes is included as Figures 4 through 7. A comparison of Friction Resistance (Sleeve Resistance), however, shows that these values do not correlate quite so well (Figures 8 through 11). To determine why there is so much variation in this value, each pressure curve was analyzed, looking at the point on the curve the chosen pressure point fell. In some tests, the pressure value corresponding to the Friction Resistance did not fall at the appropriate point on that curve. In its current form, the pressure value is stored at the time the return key is pressed, but not the time in the loop when it was chosen. A delay of only a few milliseconds as the maximum extension is reached could reflect the pressure increase that occurs as the outer test rods begin to move. This value would be too high for an accurate reading. At the time of this writing, the driver program is being modified to draw a circle on the pressure plot corresponding to the time and pressure. This will allow the operator to see when the pressure chosen occurred and allow him to modify that value.

HARDWARE

The data acquisition system has several components which include the computer, the expansion chassis, the Analog Input System board, a signal conditioning terminal panel, a power supply, and the pressure transducer. Each performs an important task in the acquisition and digitizing and interpretation of the signal.

The Compac Portable III computer was one of the first truly compact and portable micro-computers. Purchased in 1987, this unit has logged many thousands of miles throughout the State and has seen many uses. It has proven to be a durable workhorse with a 20 Megabyte hard disk, 640 Kilobytes of memory and a clock speed of 12 Mhz. It is particularly suited for field data acquisition because of its relatively small size, durability and ability to add expansion cards to its expansion chassis. This expansion chassis can be used for additional memory, hard drives, or external data acquisition, as in this case.

The expansion chassis is made by Compac Computer corporations, and provides a secure piggy-back mounting area for expansion cards. It has slots for two full sized 8/16 bit, 8 Mhz. cards and attaches to the back of the computer. It was purchased in 1989 for $140.00.

The Data Translation DT 2814 Analog Input System is a half size board, allowing oversize expansion cards to be installed adjacent to the DT 2814. This board is a 16 channel, 12 bit analog to digital converter with a 25 Khz. throughput to memory. The time for an A/D conversion is 25 micro-seconds and has a channel acquisition time of 15 micro-seconds with an accuracy of +-0.03%. It is compatible to the IBM PC/XT/AT Bus structure. It has an onboard pacer clock, but is software driven in this application. In 1989 the DT 2814 retailed for about $395.00.

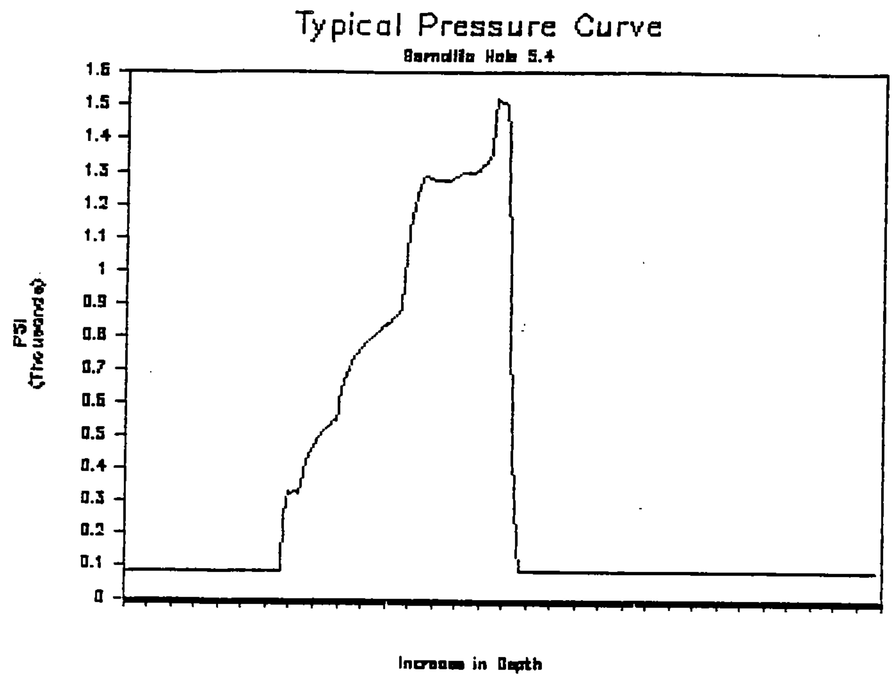
The DT 2814 is connected via a ribbon cable to a 'black box' containing a power switch, a transformer and circuit overload protection. The transformer provides DC power for the transducer, and the overload protection assures that no damage is done to components in case of a power surge. The box also contains a DT 757 screw terminal panel which provides a means of connecting the leads from the transducer to the data acquisition board. The DT 757 also provides signal conditioning. It contains an integral cable assembly terminating in a standard 20 pin, 3-M style header connector. The DT 757 retailed in 1989 for about $85.00.

The final component of the data acquisition system is an Omega PX102 pressure transducer with a 10,000 psi range maximum and two times overload capabilities. This transducer is small, shock resistant, and has an accuracy of + - 1%. The transducer takes as input five volts direct current and applies it to a Wheatstone Bridge. This bridge changes the electrical resistance by deflection of the diaphram under pressure. The transducer is mounted in-line, thus allowing the gauges to remain functional. The cost of the Omega PX102 Transducer in 1989 was about $250.00.

Other items that are needed for any field exploration operation include an electric generator to provide alternating current for the power transformer and the computer. Also useful is a line stabilizer and conditioner. A Tripplite model LC 1200 a has proven to be excellent for the application. This unit protects the computer and other electrical devices from surges of power and, to some extent, low power 'brown-outs'.

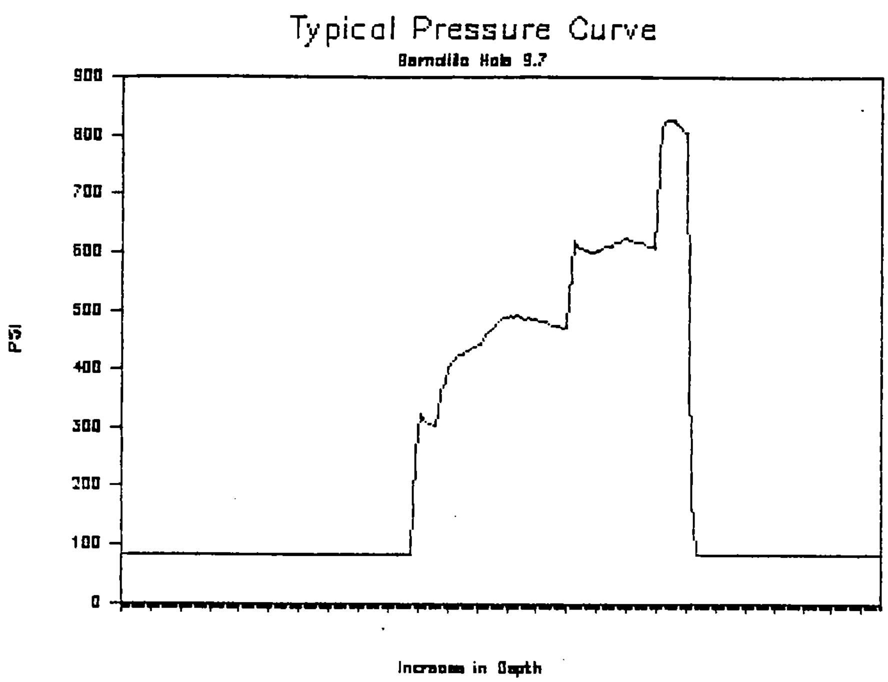
SUMMARIES AND CONCLUSIONS

The creation of the Dutch Cone Penetrometer Data Acquisition System has proven that, with a minimal investment, an older mechanical type testing device can be converted to digital output. In states that have limited areas in which DCP is usable, or for small Geotechnical consulting firms this cost savings is significant. The test results obtained thus far indicate that the calibration and accuracy of the test device is adequate, and that data for cone resistance obtained is as accurate and perhaps more reliable than that obtained from gauge readings. With additional testing and modification of the driver and data reduction methods, accuracy and reliability should improve even more. At the present time the New Mexico State Highway and Transportation Department's Geotechnical Unit is using the device in all DCP investigations for collapsible soils. Data derived from one recent investigation allowed the department to delete a Dynamic Compaction module from an active project. Test results indicated that anticipated collapsible soils were located at depths that could not be effectively treated by known technology and did not present a problem to the project. The savings on this project alone outweigh the cost of developing the DCP system.



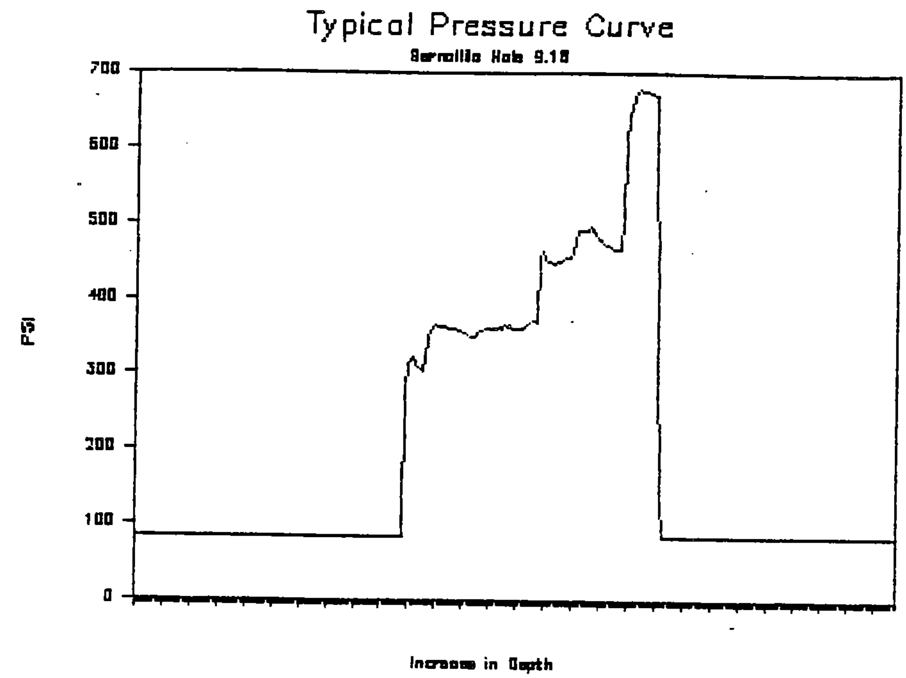
Cone Resistance at 899.9 psi
Friction Resistance at 1339.1 psi

Figure 1



Cone Resistance at 471.5 psi
Friction Resistance at 608.5

Figure 2




```
Cone Resistance at       365.5 psi
Friction Resistance at   466.7 psi
```

Figure 3

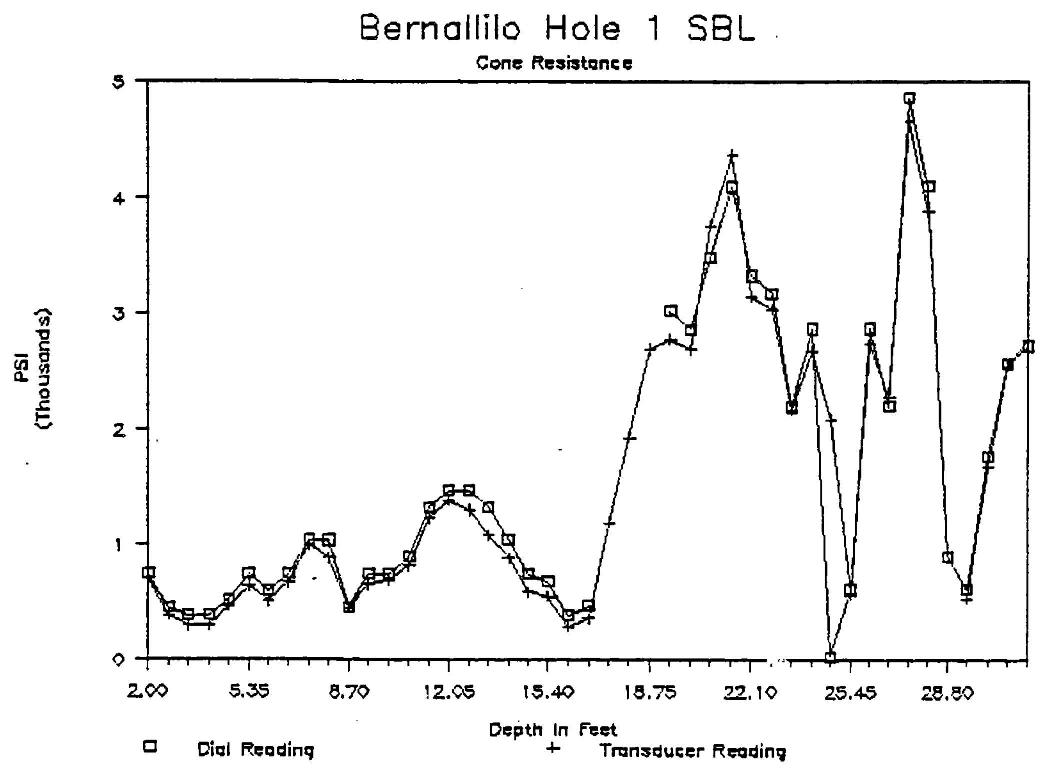


Figure 4

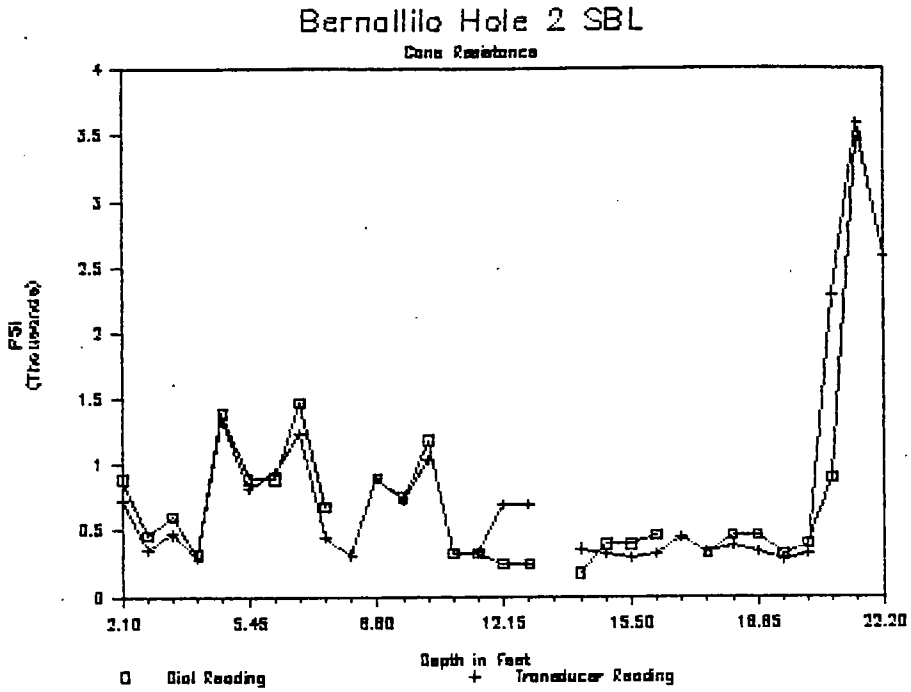


Figure 5

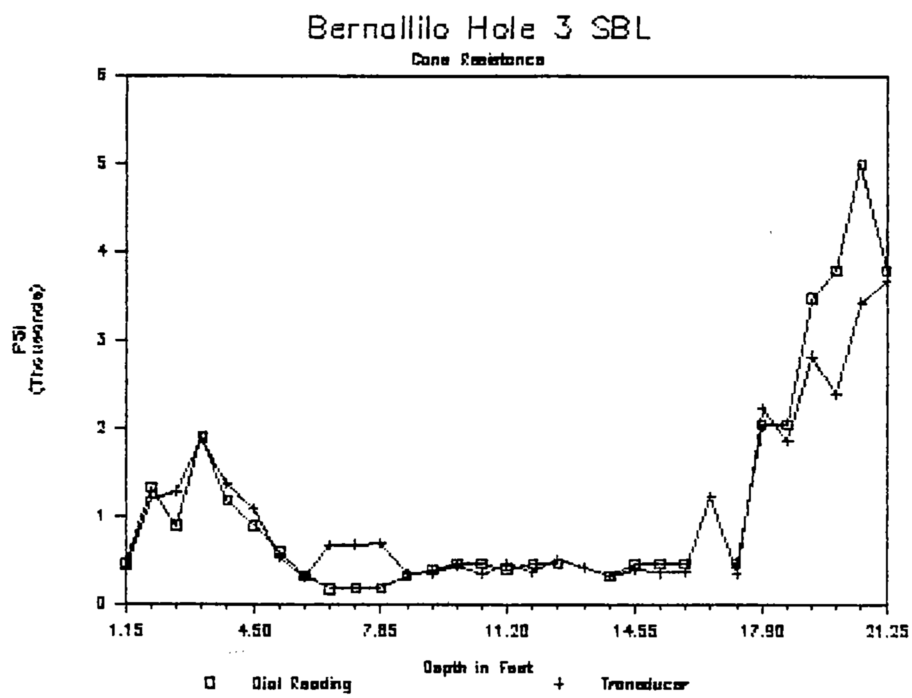


Figure 6

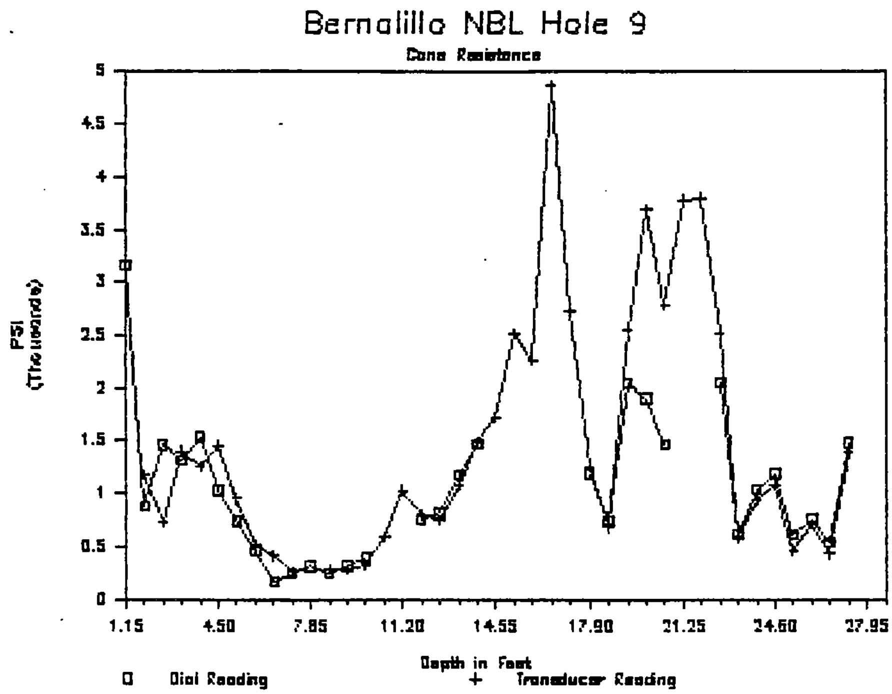


Figure 7

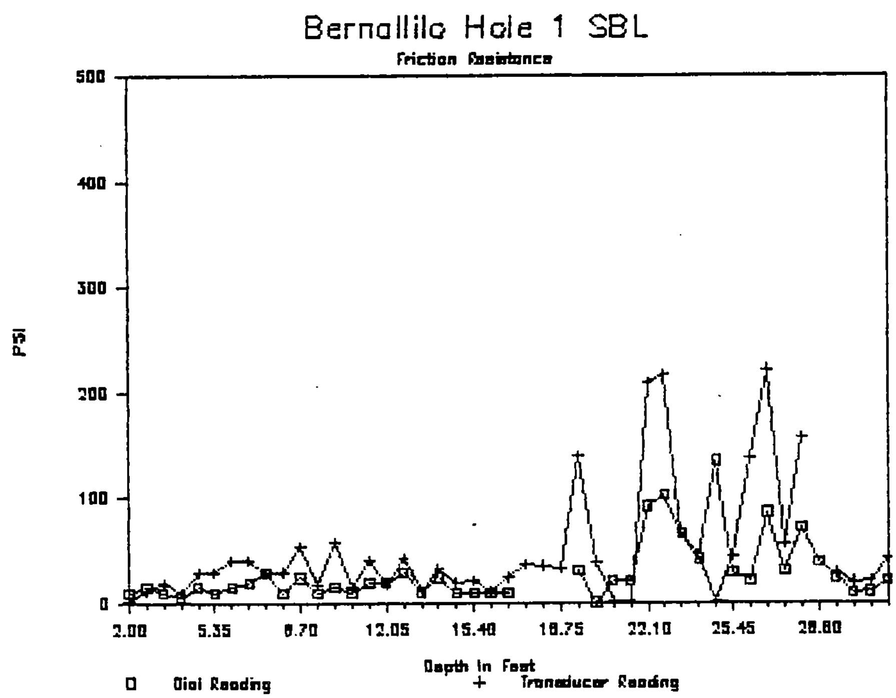


Figure 8

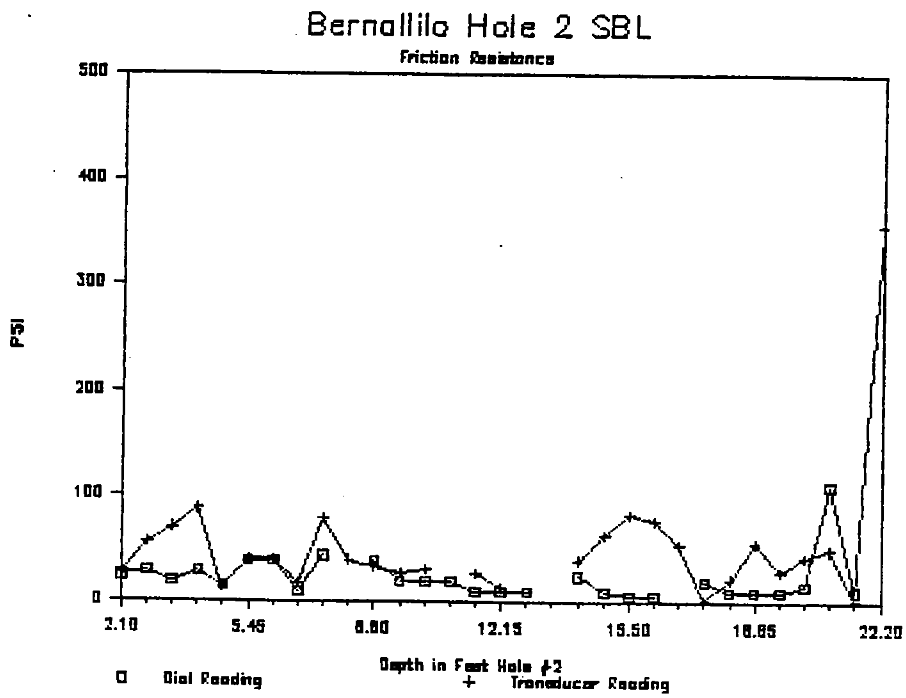


Figure 9

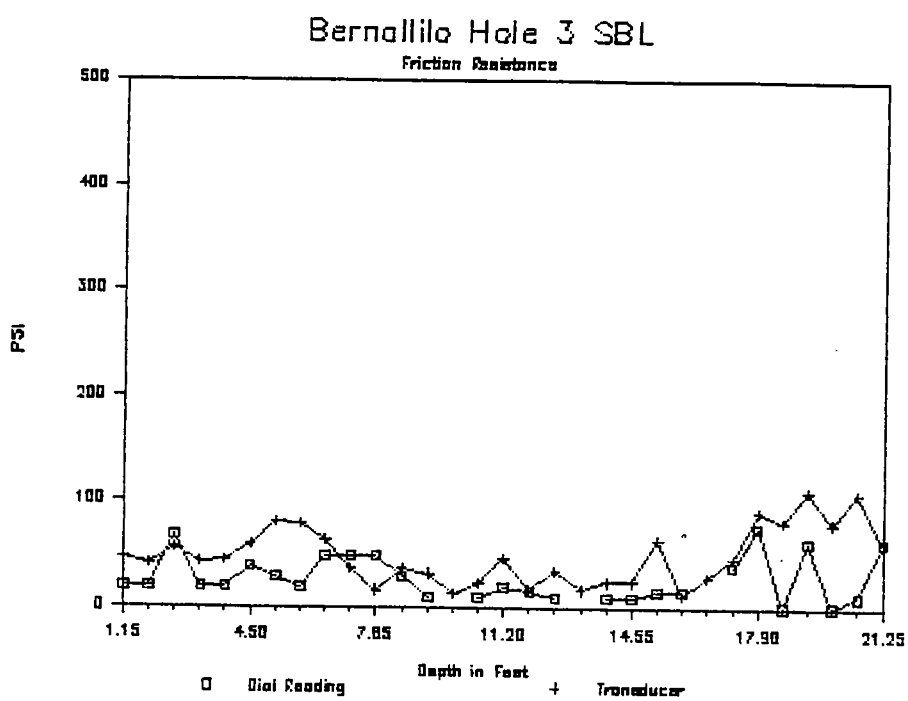
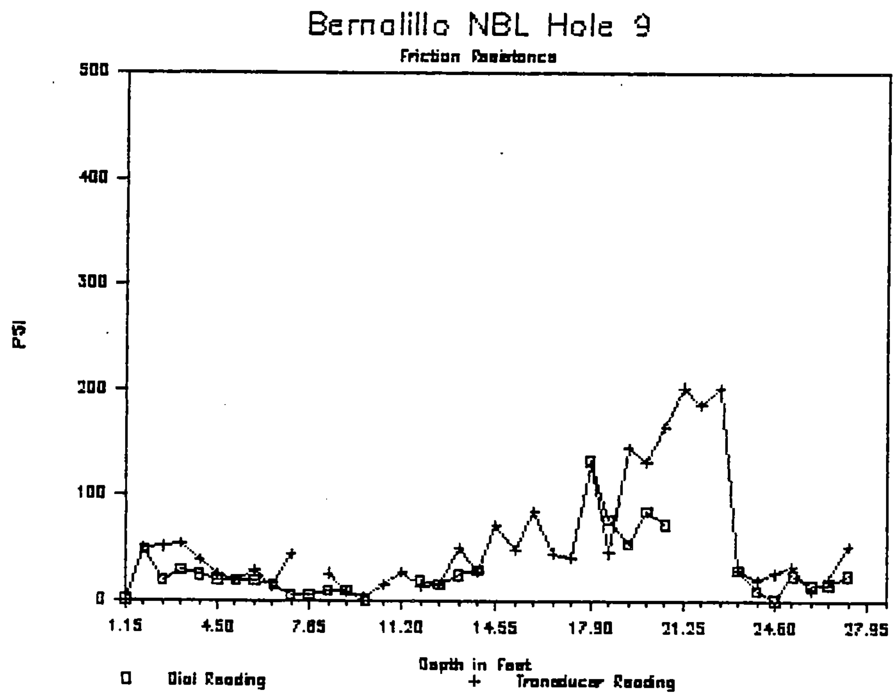


Figure 10



Bernalillo NBL Hole 9
Friction Resistance
500
400
300
200
100
0
PSI
1.15
4.50
7.85
11.20
14.55
17.90
21.25
24.60
27.95
Depth in Feet
Dial Reading
Transducer Reading

Figure 11

FLORIDA'S MINERAL AGGREGATE

CONTROL PROGRAM

W. A. Wisner, P.G.
Florida Department of Transportation

The Florida Department of Transportation (FDOT) has in place a statistically based program for controlling the quality of aggregates entering into its construction projects. This program relies on a Quality Control program implemented by materials producers and a Quality Assurance program administered by the Department, both supported by a computerized data collection and evaluation program.

Aggregate materials used in Florida include crushed limestone and granites, natural silica sands, manufactured sands, crushed concrete, artificial aggregates such as expanded shales and clay, phosphate slags and incinerator ash, riprap and road base materials such as crushed limerock, crushed cemented coquina shell, bank run shell and graded aggregate base.

At present, aggregate materials are furnished to FDOT projects from 170 mines, including nine redistribution terminals and 34 out of state sources, nine of which are located outside the United States.

Overseas sources range from phosphate slags from Newfoundland, granite from Nova Scotia, limestones from the Grand Bahamas, Jamaica and Venezuela, and crushed granite gravel from Costa Rica.

It is important at this point, to define two terms important to understanding our program. Producer Quality Control is a program developed and implemented by the producer of a material which contains the following four elements:

1. Methods to be used to control the production process from exploration of the mineral deposit through loading for shipment.

2. Sampling and testing program to determine the characteristics of the product being produced.

3. A system for collection analysis and use of test data that aids a producer in making a product of desired quality and uniformity, as well as to determine compliance of a product with the applicable specification.

4. A procedure for certifying that a product, as shipped, is in reasonably close compliance to the specification applicable to that product.

Quality Assurance is an unbiased inspection program including sampling and testing by which the Department assures that a producer Quality Control program is effectively functioning.

Under the Department's Program for Approval of Construction Aggregates, if the Department determines that the producer's Quality Control program is functioning so as to consistently produce materials which are in reasonably close conformity to specification requirements, materials are accepted by FDOT on the basis of a producer certification. The DOT does, however, reserve the right to sample and test and <u>if appropriate</u> reject obviously defective material at any point prior to it being incorporated into the work. Defective materials is generally the result of material being mishandled after leaving the mine.

This program provides several very important advantages over previously used procedures for accepting aggregates which relied heavily on sampling and testing of materials at the point of use. These include:

1. The responsibility for control of the production process and for Quality Control sampling and testing lies where it belongs, with the producer.

2. The producer's risk of having material rejected at the point of use is significantly reduced.

3. The Department of Transportation can much more effectively analyze the quality of aggregates being utilized in its projects.

This program also incorporates three very important concepts of quality control. These concepts are widely recognized as essential to effective evaluation of the characteristics of any material.

1. Variability is inherent in:

    a. Any natural occurring deposit of material.

    b. Any production process (no matter how well controlled).

    c. Sampling and testing techniques.

2. Because of this inherent variability, the test results for a <u>single</u> sample of a LOT of material is not a reasonable indication of the characteristics of the material contained in that LOT.

3. If a production process is not controlled, even a large number of samples will not reflect the characteristics of a LOT of material with a reasonable degree of certainty.

The Department's Aggregate Control Program is a dynamic program which is subject to modification to fit changes in market conditions and advances in technology. The Florida Aggregate Industry, although at first somewhat wary of accepting full Quality Control responsibility for aggregate, is fully supportive of this program. The Florida Limerock and Aggregate Institute felt so strongly about the value of this program to both the producer and the buyer, the taxpayers of our state, that they recently adopted a policy that can ultimately lead to suspension from membership of any producer who takes fraudulent actions under the program.

Prior to 1972, Florida DOT relied on method type specifications and procedures for accepting of construction aggregates consisting of:

1. At some in-state sources, DOT inspection of the production process and sampling of the entire output.

2. At out-of-state mines and some in-state mines, a geological evaluation of the deposit was made but there was no monitoring by DOT during production.

3. Sampling and testing of most material for acceptance was done at the point of use regardless of the degree of monitoring of production by DOT.

In 1972 the Department first introduced procedures which formalized control of mineral aggregates. These procedures contained:

1. A description of how sources of aggregate would be approved.

2. Criteria for establishing various levels of source approval (i.e. full, conditional, suspended, disapproved).

3. A listing of the tests to be performed on each type of aggregate material.

4. A DOT Quality Control Program based on Job Control sampling and testing verified by Progress Record sampling and testing.

In 1978, the Department began implementation of "Guidelines for Control of Limerock and Shell Base Materials" which contained the following elements:

1. The producer must develop a written Quality Control Program including a description of how this production process will be controlled and a plan for sampling and testing of the finished product. (LBR, chemical properties and Atterberg Limits, etc. at frequencies set by DOT.)

2. DOT monitoring of the mine through periodic inspection and testing to assure that the production process is in accordance with the QC plan developed by producer and approved by DOT.

3. If a producer QC Program was fully approved by DOT, acceptance of material was based on a blanket Certification covering all material furnished to a project.

4. The Department reserved the right to reject obviously defective material at any point up to it being incorporated into the work.

In 1980 the Department realized that, the data being generated by some 800 test reports issued each month could not be effectively utilized to assure the quality of aggregates entering into their projects. Some of the problems were:

1. Excessive man hours required to process and evaluate aggregate test data.

2. Delays to construction projects because test data were not available in a timely manner to those who needed it for decision making.

3. Aggregates could not be continuously identified from the mine through incorporation into a project.

4. Difficulties were being encountered in statistically evaluating test data from the producer's Quality Control program and in comparing it to test data from job-site project record samples.

5. Inability to assemble test data so that it could be used to:

   a. Analyze the effectiveness of producer QC.

   b. Develop and improve procedures for approval of aggregate sources.

   c. Studies to evaluate the reasonableness of specification limits.

To solve this problem, the Department developed a Computerized Aggregate Data Analysis System which has the following basic elements:

1. QC test data from a mine is input to the system through a computer terminal located in a nearby District Materials Office.

2. DOT Quality Assurance test data is input to the system through a computer terminal located in the District Laboratory.

3. The program assembles data in a format which allows easy comparison of all test results for a particular product.

4. The program statistically analyzes test data and, for a given set of data, calculates the mean value and standard deviation.

5. Assembled data and the statistical analysis is presented in various formats for use in areas such as:

   a. Determination of compliance of a product with the applicable specification.

b. Setting QC sampling frequencies for a particular product.

c. Studies to determine the reasonableness of specification limits.

d. Studies to develop aggregate source approval procedures.

Additional features of the computer program can be found listed in Figure 1.

In 1981 Florida DOT, on the basis of our experience with aggregate quality control systems and the conclusions which could then be drawn from our Computerized Aggregate Analysis System, implemented a Standard Operating Procedure (S.O.P.) for Evaluation, Approval and Control of Coarse Aggregate Sources. In 1983, Florida DOT developed an S.O.P.to cover Fine Aggregate. By 1986 several Aggregate Redistribution Terminals had been established in Florida and the S.O.P.'s were expanded to provide Quality Control requirements for aggregates passing through these terminals. An S.O.P. covering base materials was also developed during this time.

This past year, DOT combined its various Standard Operating Procedures into a Department rule for Approval of Aggregate Sources. We also:

1. Modified the scope statements to provide a better understanding of the purpose of the program.

2. Strengthened their ability to effectively administer the program.

3. Revised requirements for redistribution terminals.

4. Reduced the variability allowed in production of aggregates used in hot bituminous mixes.

5. Added time limits for availability of quality control test results.

6. Added legal notification and administrative hearing provisions required by the Florida Administrative Procedures Act.

The basic elements of the <u>current program</u> are:

A. Mines and Redistribution Terminals are assigned classifications.

Mines are classified into three types in accordance with the ability of DOT to monitor the producer's Quality Control program. Mines located remote from a DOT Materials Laboratory cannot be monitored as frequently, so more stringent Quality Assurance is applied and limitations are placed on the producer's certification. Redistribution terminals are classified into three types in accordance with whether or not the terminal and the mine from which it is receiving materials are in the same ownership, and the type of mine from which materials are being received.

B. Prior to seeking Source Approval, a producer must:

1. Develop a deposit and evaluate it to ascertain that the minerals contained therein can be processed to meet specification requirements.

2. Establish a processing facility and process controls which will produce materials meeting the quality and uniformity requirements for each product for which approval is sought.

3. Demonstrate through a series of tests on the finished product that each product to be produced will have a greater than a 95% probability of complying with all applicable specification requirements and will meet any uniformity requirements set up in DOT procedures. The minimum number of samples required for initial source approval are set out in the DOT Mineral Aggregate Manual.

4. Develop and obtain DOT approval of an individualized written Quality Control program containing the following elements:

   a. A description of the physical location of the mine.

   b. A Production Flow Diagram---which is a step by step description of mining and processing operations.

   c. A means of providing positive identification of the individual product contained in each stockpile.

   d. A description of the procedure to be used to assure that products are loaded into only clean hauling units and each load actually contains the product indicated on the shipping ticket.

   e. A statistically based sampling plan showing the method of obtaining product QC samples, including sampling devices and locations at which QC samples will be obtained. (DOT will assign QC sampling frequencies per production LOT at the time of source approval).

   f. Method to be used to maintain records of QC test data and to perform statistical analysis of this data to assess compliance with specification requirements.

   g. Identification of the personnel responsible for production Quality Control and the person who has overall responsibility for Quality Control.

   h. A plan for dealing with failures in the production process which are indicated by QC tests.

i. A description of the laboratory facilities to be used for QC testing. The laboratory must meet standards established by DOT.

j. Designation of the technicians who will perform QC tests and a list of their qualifications. Some technicians must be certified by DOT.

k. Designation of the turn-around-time for QC test results to be available at producer's QC office and to be submitted to DOT inspection personnel.

C. Upon receiving a request for approval of an aggregate source the DOT Aggregate Control Engineer conducts an evaluation of the source consisting of the following actions:

1. A geologic evaluation of the parent material to assure that it does not contain minerals which may adversely affect performance of the construction items in which the aggregate will be used.

2. Inspection of mining and processing operations.

3. Inspection of the producer's laboratory facilities.

4. Review of the QC Program submitted by the producer.

5. Statistical analysis of the producer's test data submitted for initial approval. This analysis must substantiate that aggregate produced will have a greater than 95% probability of complying with the applicable specification.

D. If DOT finds that the source is eligible for approval they:

1. Assign a Mine Number.

2. Designate the specific products for which source approval is granted.

3. Assign sampling frequencies for the producer's QC testing based on their statistical analysis of the producer's initial test data.

4. Approve the source at the Conditional level of Approval.

5. Charts for assigning QC sampling frequencies are contained in the DOT Mineral Aggregate Manual.

E. After a mine is approved, DOT monitors the effectiveness of the producer's QC Program through its Quality Assurance Program which consists of the following basic elements:

1. Plant inspections, generally at a frequency of once per week.

2. Product sampling and testing in conjunction with plant inspections.

3. Some sampling and testing at the point of use.

4. Action as the result of findings of inspections and testing which may result in:

   a. A change in the assigned QC sampling frequency for one or more products produced at that mine.

   b. A change in the level of source approval for the mine or for one or more of the products produced at the mine.

F. The program is set up with four levels of source approval:

FULL APPROVAL STATUS - Producer may certify material directly to DOT projects.

CONDITIONAL APPROVAL STATUS - Limitations are placed on the producer's right to certify directly to DOT projects.

SUSPENDED STATUS - The producer is restricted from shipping one or more products to DOT projects for a period of at least 30 days. A source may not be in this status for more than 90 days.

REVOKED STATUS - Producer is restricted from shipping one or more products to DOT projects for a period of not less than 6 months.

The level assigned depends on:

1. The probability of producing products which are in reasonably close conformity to specifications and which meet the special uniformity requirements established for certain products, as determined from a statistical analysis of the producer's QC test data.

2. DOT analysis, through its Quality Assurance program, as to how well the producer's QC Program is functioning.

3. Demonstrated ability by the producer to consistently control the quality of the products being produced. A mine is not allowed to continuously fluctuate between the various levels of approval.

4. An indication that the producer is not maintaining accurate records in conjunction with his QC Program may cause DOT to place a mine in Suspended Status.

5. An indication of producer falsification of records or fraudulent certification of aggregates may cause DOT to place a mine in Revoked Status.

None of this would be worth the effort if there weren't numerous benefits associated with a well designed Quality Control/Quality Assurance Program. These include:

A. Definite responsibilities can be assigned to the producer for process control and to the Department for Quality Assurance and Acceptance.

B. Specifications which use statistical concepts can clarify the acceptance and rejection points.

C. QC/QA specifications are more legally defensible, with clearly defined responsibilities and acceptance criterion.

D. Well written specifications and operating procedures which allow and encourage the producer to be innovative, should result in more efficiency and thus lower costs. In the long term this should also increase the quality of the finished product.

E. The Department can make more efficient use of inspection and testing personnel by relying on statistics.

F. This program has had a side benefit of providing a very effective means of improving communication between the FDOT and the aggregate industry.

G. The data bank in the Computerized Aggregate Analysis System is a very useful tool in problem solving and research because data on the as-produced characteristics of various aggregate products is readily available in a convenient format.

H. The quality and uniformity of aggregates produced in Florida has clearly improved.

Today, the Florida Department of Transportation is working in conjunction with Florida aggregate industry, on a very important issue.

A recent FHWA Technical Advisory titled "Acceptance of Materials" treats aggregates as a project Produced Material rather than a Manufactured Material. The TA defines Project Produced Materials as materials subject to changes during transportation from the plant and by subsequent manipulation that occurs at the project site. The problem here is that the TA requires that acceptance sampling of Project Produced Materials should be performed at the last possible point before incorporation into the project or at the last point that the test can be performed. Conducting all sampling for acceptance at the project site negates many of the benefits of a statistically based QC Program.

It is our position that: (1) aggregates are manufactured under controlled conditions; (2) only certain properties are subject to change in characteristics after leaving the plant; and (3) these changes may not be too significant. We feel we have more effective control of materials under our present program. It is also important to reiterate here that; (1) the test results on samples of a material are meaningful only when the process under which the material is produced is under control; and (2) a single sample does not effectively indicate the properties of a LOT of a material.

In concluding, I would like to summarize some of the concepts and philosophies of the Department.

1. Quality and control of materials are established at the time of production.

2. The responsibility for quality and the control of the production process lies with the producer.

3. Sampling and testing do not determine quality, but determines with some probability, whether the material meets specification requirements.

4. All acceptance plans carry some risk to both the Department and the contractor that a product will be either accepted or rejected in error. These risks must be identified and managed according to the material being tested and the consequences of failure.

5. The use of statistics is an important part of a Quality Control Quality Assurance Program. Statistics are a tool which help in:

    a. Establishing concise quality levels.

    b. Developing valid tolerances for specification limits.

    c. Evaluating test reports and measurements.

    d. Assigning responsibilities.

6. Quality cannot be inspected or compiled and tested into a product, it must be designed and constructed into the product.

7. The bottom line is that it is too late to determine the quality of materials at the point of use.

FLORIDA DEPARTMENT OF TRANSPORTATION
AGGREGATE CONTROL PROGRAM

SECURITY

- READ DATABASE - ANYONE WITH ACCESS TO FDOT MAINFRAME SYSTEM.
- INPUT TEST DATA - ONLY THOSE AT THE DISTRICT MATERIALS OFFICE AND BRANCH LABORATORIES RESPONSIBLE FOR DATA INPUT AND STATE MATERIALS OFFICE, GAINESVILLE.
- UPDATE AND CORRECT TEST DATA - ONLY THOSE AT THE DISTRICT MATERIALS OFFICE AND BRANCH LABORATORIES RESPONSIBLE FOR DATA INPUT AND STATE MATERIALS OFFICE, GAINESVILLE.
- CHANGE MINE OR PRODUCT STATUS - ONLY AGGREGATE CONTROL UNIT PERSONNEL, STATE MATERIALS OFFICE, GAINESVILLE.
- CHANGE CODES TABLE - ONLY AGGREGATE CONTROL UNIT PERSONNEL, STATE MATERIALS OFFICE, GAINESVILLE.

FEATURES

- COMPLETE DATA LISTING AND ANALYSIS ON SPECIFICATION REQUIREMENTS OVER ANY SPECIFIED TIME PERIOD OF PRODUCTION. SIEVE RANGES, LA ABRASION, -200, CARBONATE CONTENT, LIMEROCK BEARING RATIO, ETC...
- SOURCE INFORMATION LISTING - CONTACT PEOPLE, QC LABORATORY, LOCATION, APPROVED PRODUCTS, SAMPLING FREQUENCIES, ETC...
- PRINTED REPORTS
- DATA MAINTENANCE - APPROVE AND DISAPPROVE MINES, PRODUCTS, AND CODES.
- REJECTION OF TEST DATA FROM A NONAPPROVED MINE OR FOR A NONAPPROVED PRODUCT. (DESIGN MIXES)

OTHER DATA EXTRACTION AND ANAYSIS PROGRAMS

- SAS(STATISTICAL ANALYSIS SYSTEM)
  - ANALYSIS OF THE LAST 30 QC TEST RESULTS FOR SAMPLING AND TESTING EVALUATION.
  - ABBREVIATED LISTING OF SOURCE INFORMATION AND DATA.
  - SPECIAL ANALYSES AND DATA PLOTTING FOR TREND ANALYSIS.
  - LISTING OF NON-ROUTINE DATA FOR RESEARCH AND INFORMATION PRUPOSES.
  - EXTRACTION OF DATA FOR DOWNLOADING.
- PC(PROGRAMS)
  - SPECIAL ANALYSIS AND DATA PLOTTING.

Figure 1. Aggregate Control Program Features.

DATE PRINTED 03/07/90

FLORIDA DEPARTMENT OF TRANSPORTATION
BUREAU OF MATERIALS AND RESEARCH

COARSE AGGREGATE ANALYSIS SYSTEM

DATE FROM 01/01/89 DATE TO 12/31/89

MINE/TERM NUM 08005 MINE NUM 08005
AGGREGATE GRADE ASTM OR FLA GRADE 57 DOT CODE 12
MATERIAL TYPE CRUSHED STONE
REQUIRED SIEVE SIZES 1.0IN 3/4IN 3/8IN NO. 4 NO. 8
SPECIFICATION REQUIREMENTS 100 90-100 20-55 0-10 0-5
TYPE OF SAMPLE GC PRODUCER AT QUARRY (PRIMARY)

DATE TESTED SIEVE SIZES	SIEVE NO-1 1.0IN	SIEVE NO-2 3/4IN	SIEVE NO-3 3/8IN	SIEVE NO-4 NO. 4	SIEVE NO-5 NO. 8	SIEVE NO-6	SIEVE NO-7	% PASS 200	LA ABRA	SAMPLE NUMBER
01-03-89	100	100	37	5	3			1.17	37	10189
01-03-89	100	100	32	5	4			.76		20189
01-04-89	100	100	61	6	4			.89		30189
01-04-89	100	100	61	7	4			.92		40189
01-05-89	100	100	32	4	3			.93		50189
01-05-89	100	100	40	6	4			.92		60189
01-06-89	100	100	35	5	4			.75		70189
01-05-89	100	100	29	3	2			.90		80189
01-09-89	100	100	37	3	3			.97		10289
01-09-89	100	100	29	3	2			.93		20289
01-10-89	100	100	34	4	4			.94		30289
01-10-89	100	100	31	5	4			.65		40289
01-11-89	100	100	36	5	4			.59		5028
11-13-89	100	100	35	6	5			1.07		246
11-22-89	100	100	36	7	4			.89		1478
11-22-89	100	100	39	5	4			1.39		24789
11-27-89	100	100	50	7	3			.91		14889
11-27-89	100	100	35	5	5			.87		24889
12-04-89	100	100	30	3	3			.91		14989
12-04-89	100	100	37	5	4			.90		24989
12-11-89	100	100	39	4	4			.94		25089
12-16-89	100	100	30	5	4			.99		15189
12-18-89	100	100	27	4	4			1.06		25189
12-26-89	100	100	35	4	4			.95		15289
12-26-89	100	100	39	5	4			1.04		26289
N-COUNT	352	352	352	352	352			352	7	
AVERAGE	100.0	100.0	33.7	4.1	3.3			.95	36.3	
STD DEV	0.0	0.0	4.4	1.0	0.8			.161	2.6	
COEF VAR	0.0	.0	13.0	24.3	24.2			16.9	7.1	
EST FAILURES	0	0	0	0	1			0	0	

Figure 2. Computer Printout from Aggregate Control Program.

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FLORIDA DEPARTMENT OF TRANSPORTATION
BUREAU OF MATERIALS AND RESEARCH

COARSE AGGREGATE ANALYSIS SYSTEM

DATE FROM 01/01/89 DATE TO 12/31/89

MINE/TERM NUM 08005 MINE NUM 08005
AGGREGATE GRADE ASTM OR FLA GRADE 67 DOT CODE 12
MATERIAL TYPE CRUSHED STONE
REQUIRED SIEVE SIZES 1.0IN 3/4IN 3/8IN NO. 4 NO. 8
SPECIFICATION REQUIREMENTS 100 90-100 20-55 0-10 0-5
TYPE OF SAMPLE PRODUCER (1/2) OF SPLIT SAMPLE

DATE TESTED SIEVE SIZES	SIEVE NO-1 1.0IN	SIEVE NO-2 3/4IN	SIEVE NO-3 3/8IN	SIEVE NO-4 NO. 4	SIEVE NO-5 NO. 8	SIEVE NO-6	SIEVE NO-7	% PASS 200	LA ABRA	SAMPLE NUMBER
01-17-89	100	100	41	7	5			1.16		30389
08-28-89	100	100	38	5	4			1.56		13589
10-23-89	100	100	33	4	4			1.39		14389
12-11-89	100	100	33	5	4			2.16	40	15089
N-COUNT	4	4	4	4	4			4		
AVERAGE	100.0	100.0	36.3	5.3	4.3			1.57		
STD DEV	0.0	0.0	3.9	1.3	0.5			.428		
COEF VAR	0.0	.0	10.7	24.5	11.6			27.2		
EST FAILURES	0	0	0	0	7			33		

DATE PRINTED 03/07/90

FLORIDA DEPARTMENT OF TRANSPORTATION
BUREAU OF MATERIALS AND RESEARCH

COARSE AGGREGATE ANALYSIS SYSTEM

DATE FROM 01/01/89 DATE TO 12/31/89

MINE/TERM NUM 08005 MINE NUM 08005
AGGREGATE GRADE ASTM OR FLA GRADE 67 DOT CODE 12
MATERIAL TYPE CRUSHED STONE
REQUIRED SIEVE SIZES 1.0IN 3/4IN 3/8IN NO. 4 NO. 8
SPECIFICATION REQUIREMENTS 100 90-100 20-55 0-10 0-5
TYPE OF SAMPLE D O T (1/2) OF SPLIT SAMPLE

DATE TESTED SIEVE SIZES	SIEVE NO-1 1.0IN	SIEVE NO-2 3/4IN	SIEVE NO-3 3/8IN	SIEVE NO-4 NO. 4	SIEVE NO-5 NO. 8	SIEVE NO-6	SIEVE NO-7	% PASS 200	LA ABRA	SAMPLE NUMBER
01-03-89	100	100	39	4	3			1.16	36	10189
04-17-89	100	99	26	4	3			1.03		11689
08-28-89	100	100	34	4	3			1.41		13589
10-23-89	100	100	33	4	4			1.43		14389
12-11-89	100	100	32	[illegible]	4			1.68	38	15089
N-COUNT	5	5	5	5	5			5	2	
AVERAGE	100.0	99.8	32.8	4.0	3.4			1.34	37.0	
STD DEV	0.0	0.4	4.7	0.0	0.5			.254	1.4	
COEF VAR	0.0	.4	14.3	.0	4.7			18.9	3.7	
EST. FAILURES	0	0	0	0	0			5	0	

Figure 3. Computer Printout of Split Samples for Comparison.

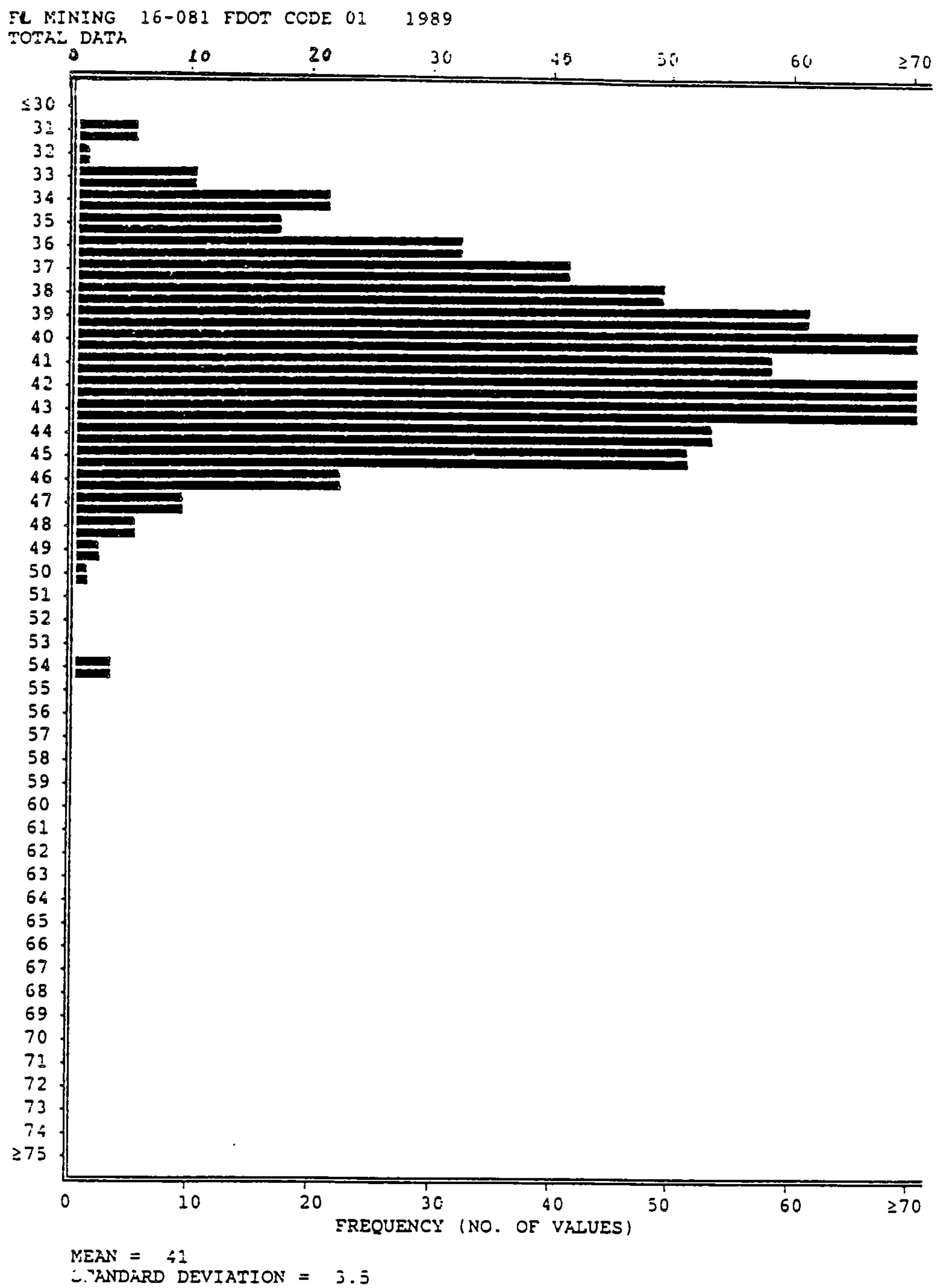


Figure 4. Histogram of Distribution of Data for Number 30 Sieve for Silica Sand.

Figure 5. Histogram on Number 30 Sieve for Silica Sand(Producer Half of Split Sample).

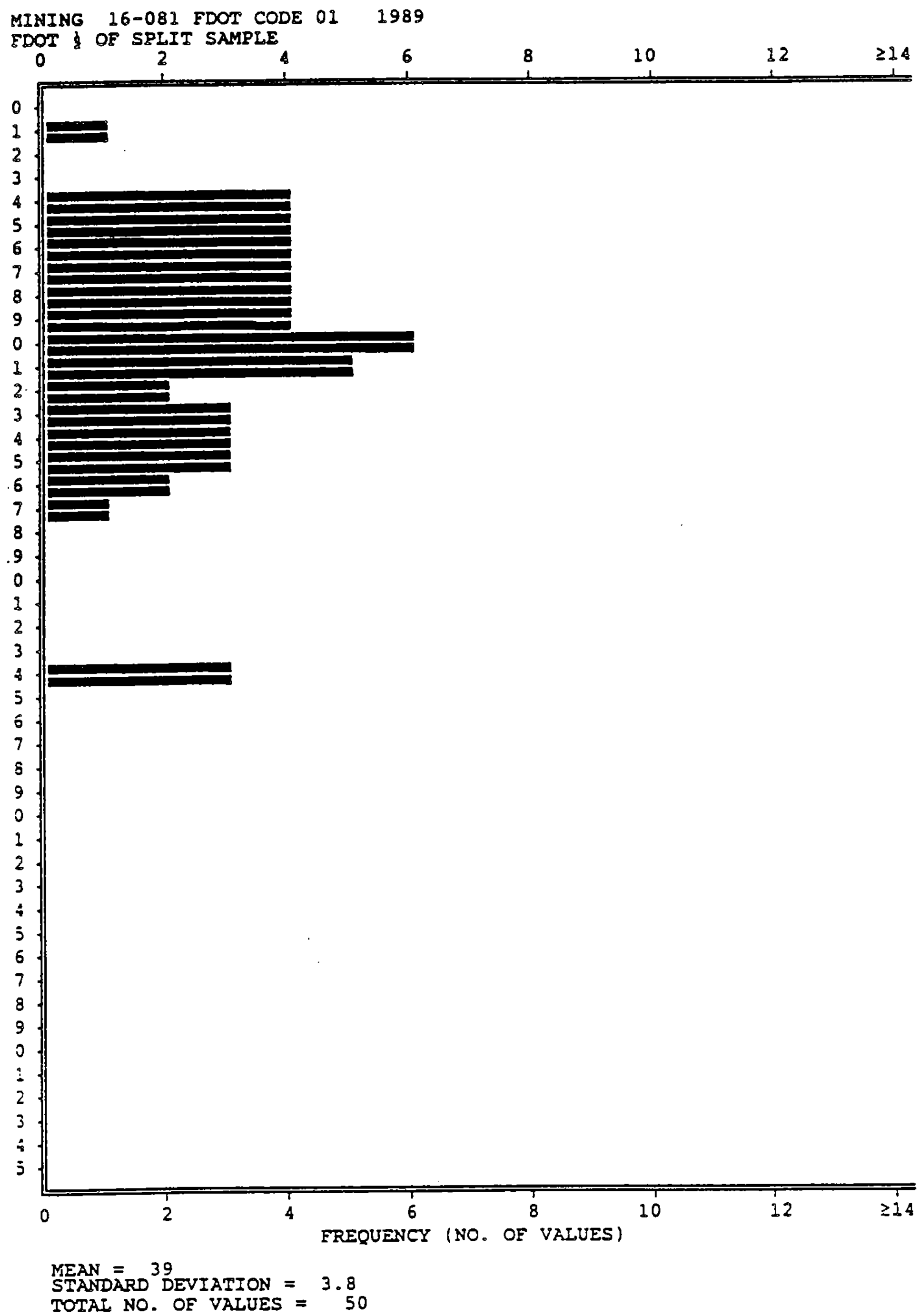
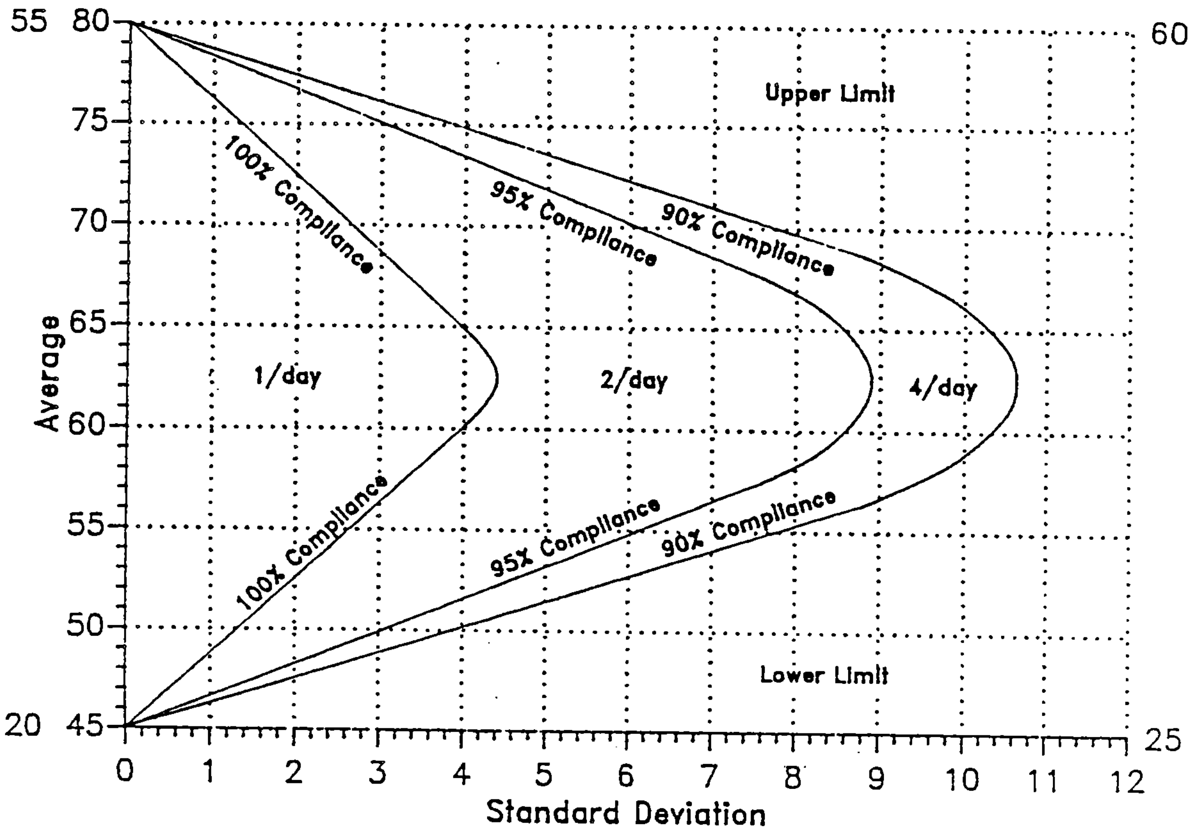


Figure 6. Histogram on Number 30 Sieve for Silica Sand(FDOT Half of Split Sample).

Illustration of Minimum Sampling and Testing Frequencies for Aggregates with a 35% Variation Between Upper and Lower Sieve Limits



Appendix 7

Figure 7. Sampling and Testing Frequency Chart for a Double-Ended Specification.

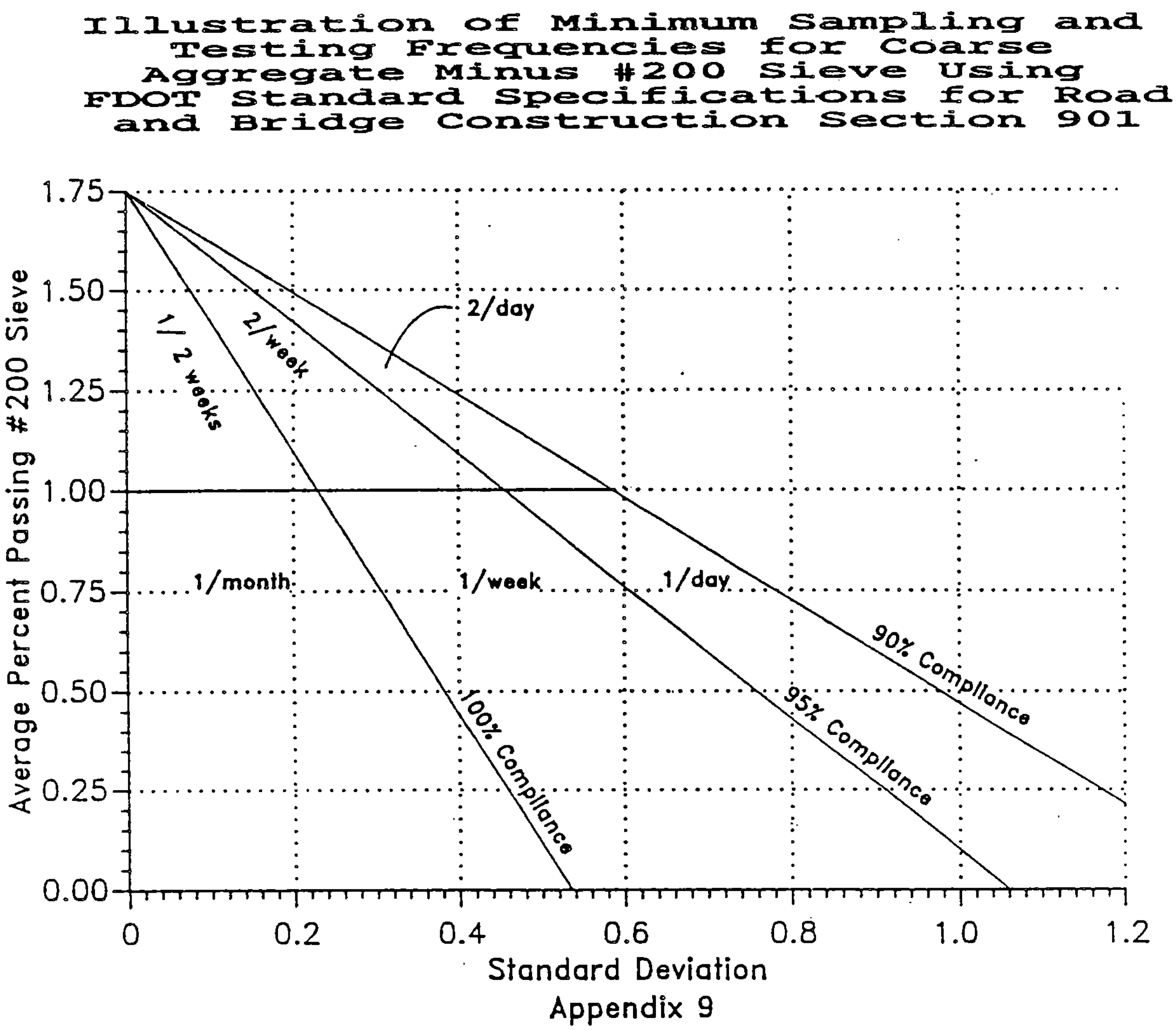


Figure 8. Sampling and Testing Frequency Chart for a Single-Ended Specification.

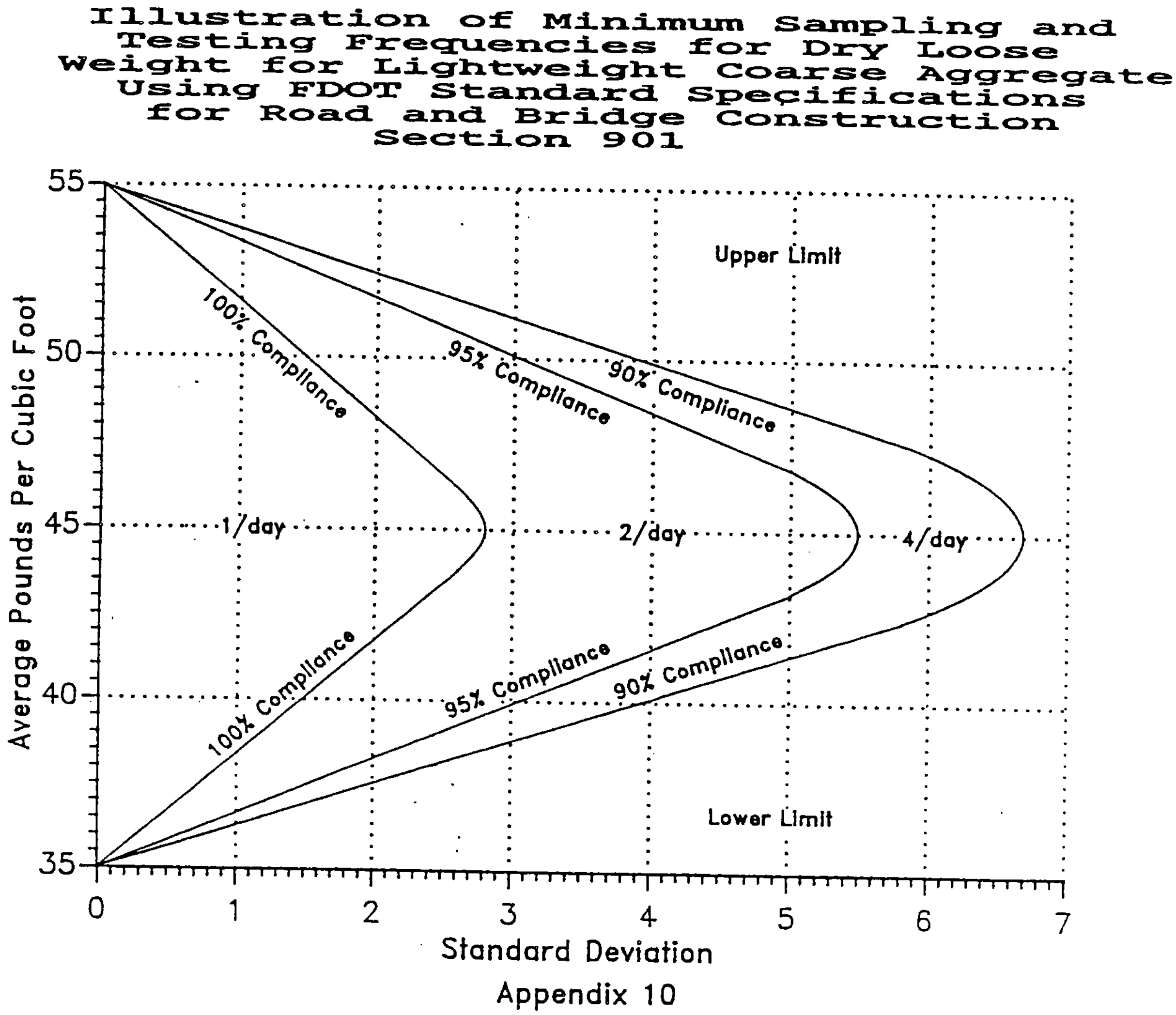


Figure 9. Sampling and Testing Frequency Chart for Specialized Lightweight Aggregate.

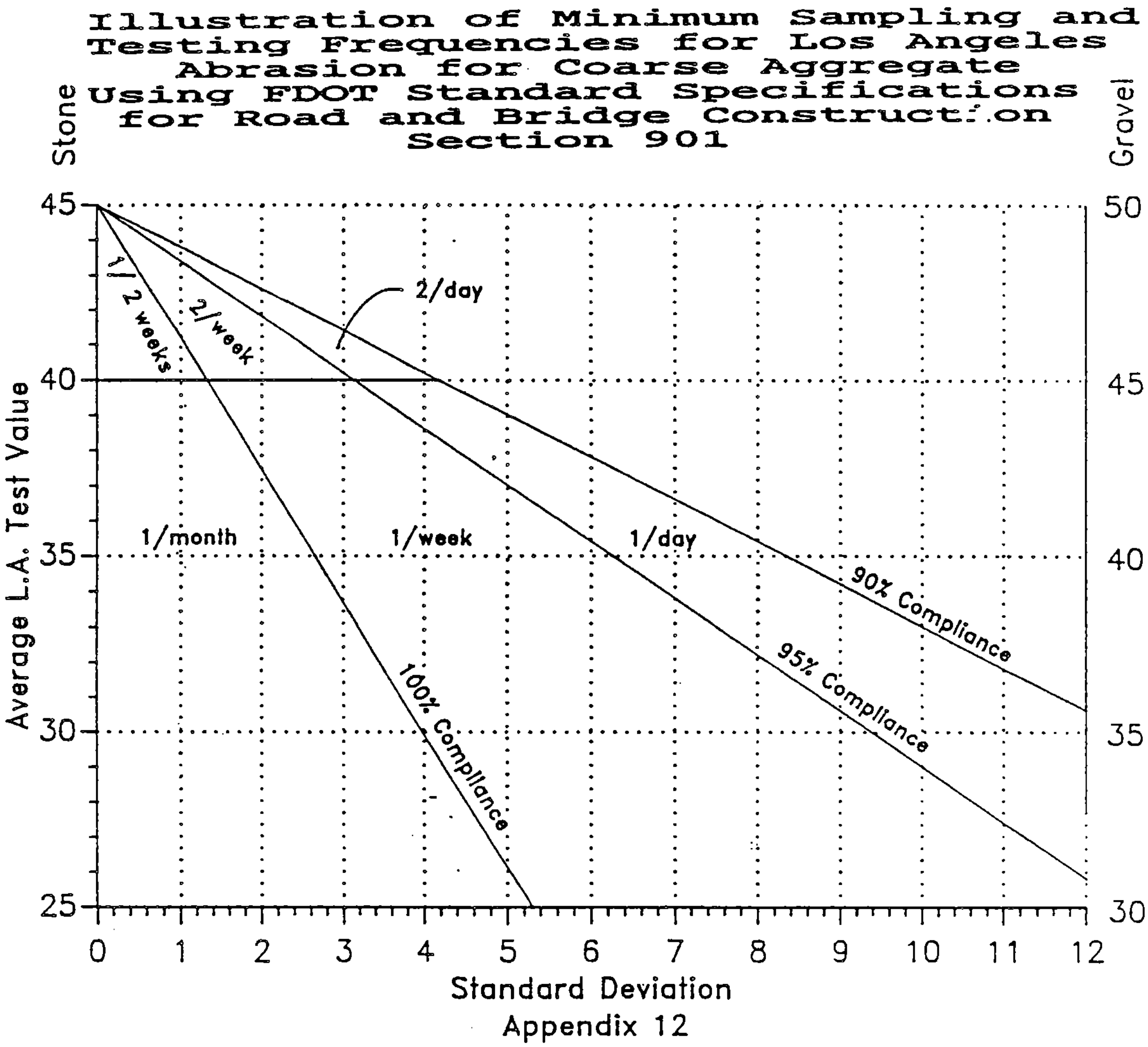


Figure 10. Sampling and Testing Frequency Chart for L. A. Abrasion of Different Types of Aggregate.

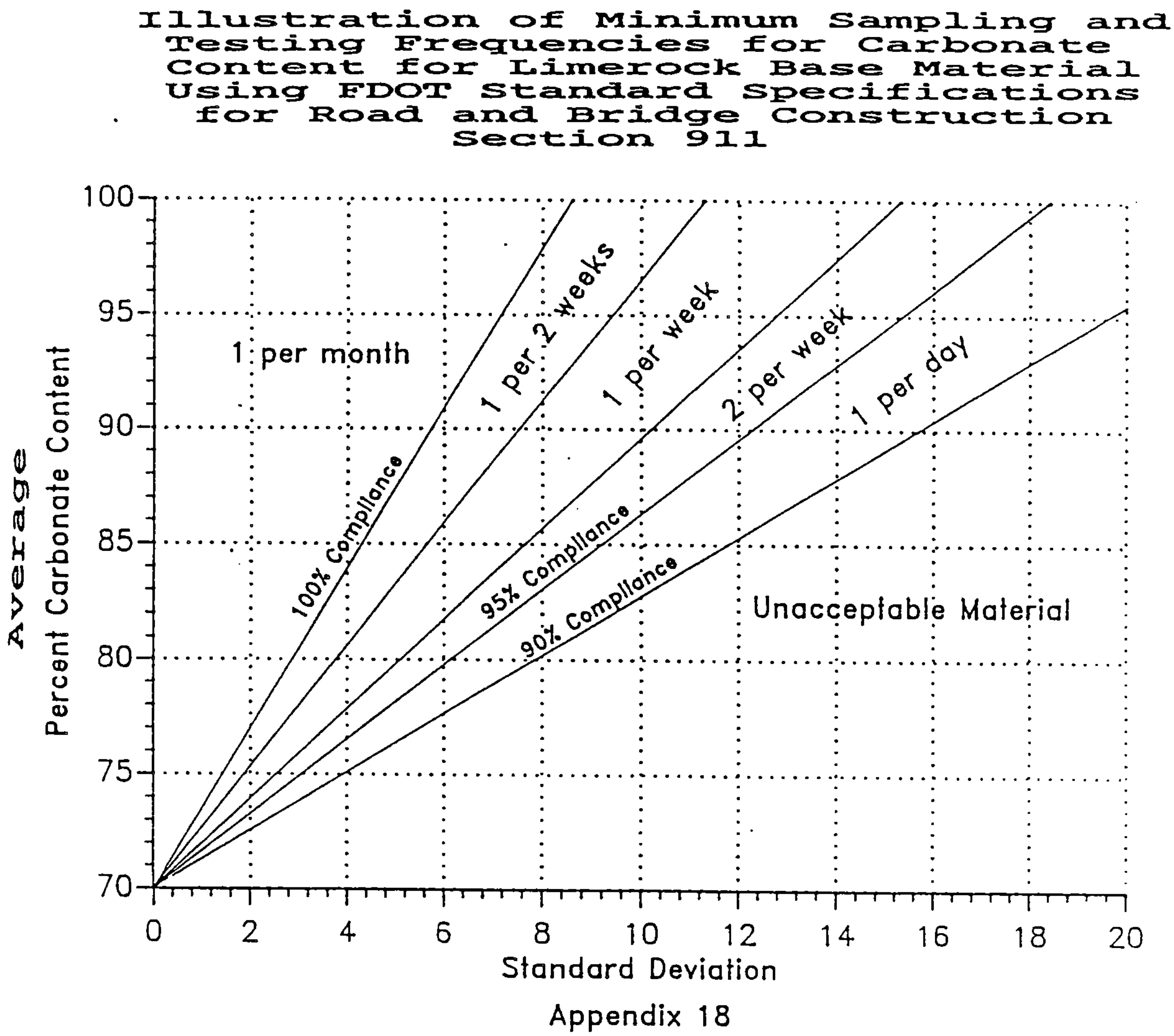


Figure 11. Sampling and Testing Frequency Chart for Carbonate Content in Limerock Base Material.

Illustration of Minimum Sampling and Testing Frequencies for Limerock Bearing Ratio for Base Materials Using FDOT Standard Specifications for Road and Bridge Construction Sections 911, 913, and 915

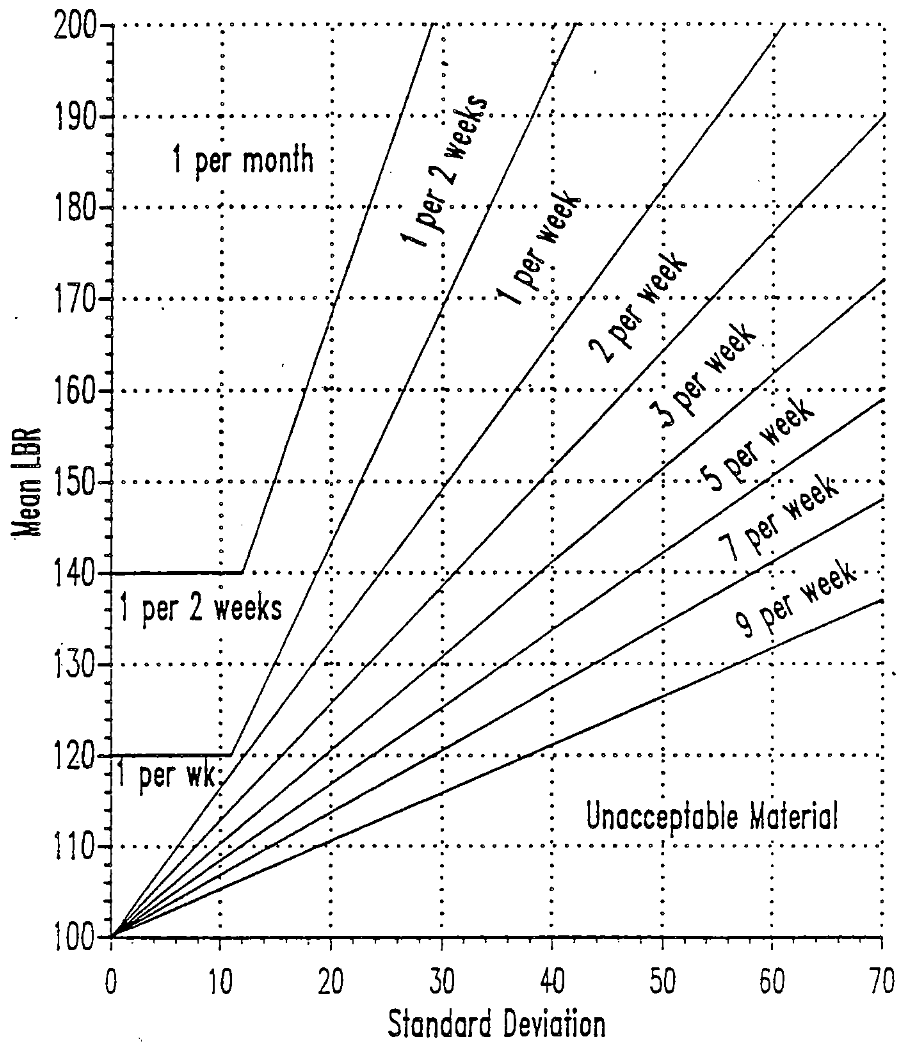
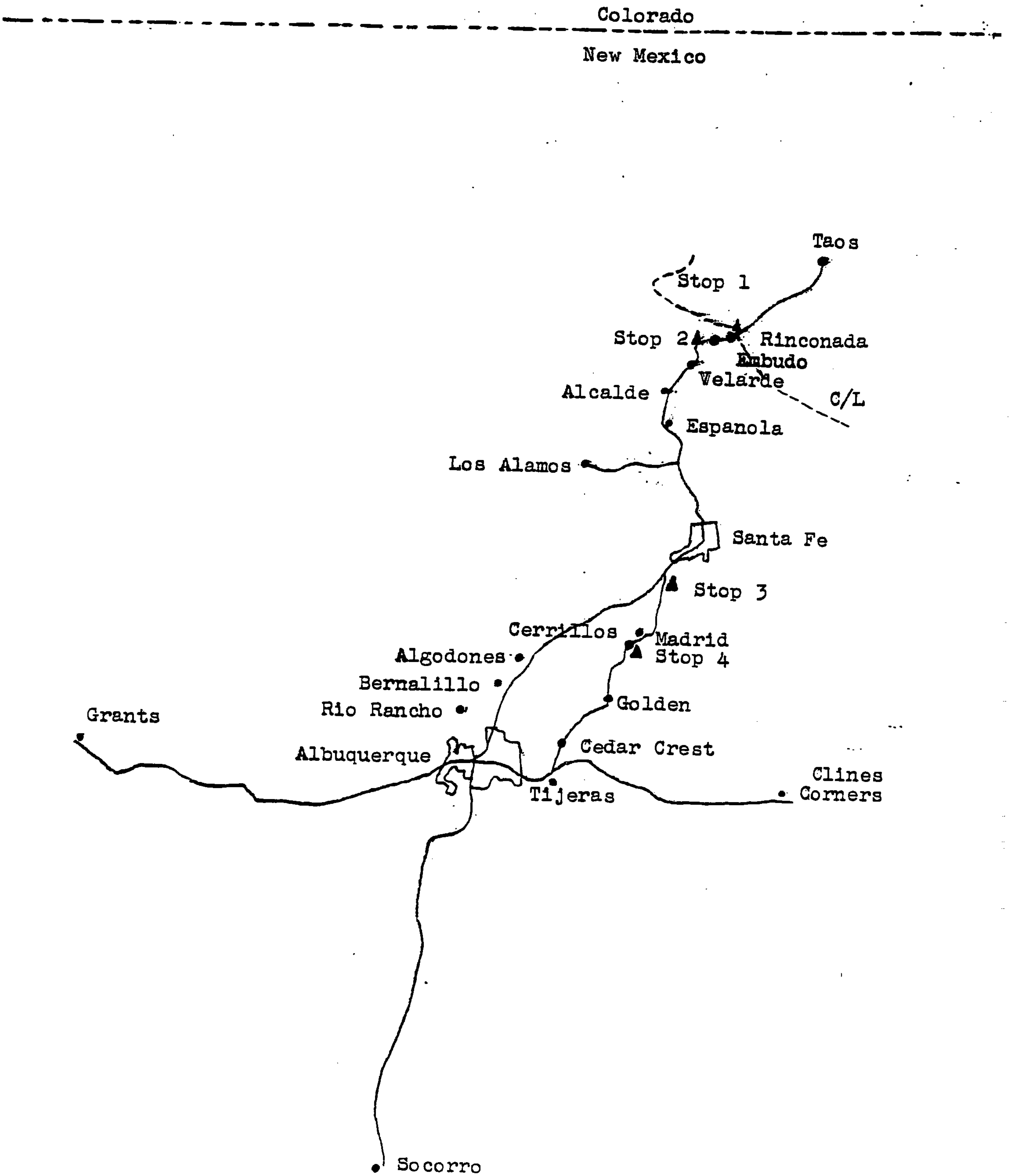


Figure 12. Sampling and Testing Frequency Chart for Limerock Bearing Ratio Test in Base Materials.

ROAD LOG OF FIELD TRIP



Colorado
New Mexico
Taos
Stop 1
Stop 2
Rinconada
Embudo
Velarde
C/L
Alcalde
Espanola
Los Alamos
Santa Fe
Stop 3
Cerrillos
Madrid
Stop 4
Algodones
Bernalillo
Golden
Rio Rancho
Grants
Cedar Crest
Albuquerque
Clines Corners
Tijeras
Socorro

41ST ANNUAL HIGHWAY GEOLOGY SYMPOSIUM

ROAD LOG of FIELD TRIP

From Albuquerque to the Rio Arriba/Taos County Line (1.2 miles north of Rinconada), via Santa Fe, Espanola, and Embudo; and return to Albuquerque via N.M. 14 (south of Santa Fe).

Daryl R. Busch

Assembly Point: Ramada Hotel Classic, corner of Louisiana and Menual, Albuquerque, NM.
Departure Date & Time: August 16, 1990 at 8:00 a.m.
Distance: 230.0 miles
Stops: 4

Cumulative Mileage	Mile Post	Comment
0.0		Leave parking lot and turn left (south) onto Louisiana Blvd. Proceed through intersection.
0.05		On the right. Coronado Mall, largest shopping center in Albuquerque area.
0.5		On the left. Winrock Center, second largest shopping center.
0.6		Turn right to get on Interstate 40.
0.9	162.6	Merge with I-40 traffic.
2.8	160.7	Straight ahead. West Mesa Volcanoes, a string of north trending volcanoes, spatter cones, and basalt-capped volcano cliffs.
3.4	160.1	Straight ahead. Rio Grande Valley (Spanish for "large river"), which lies within the Basin and Range Province. The Rio Grande Rift cuts New Mexico in half.
3.8		Exit right at the Big-I (intersection of I-40 with I-25) to proceed northward to Santa Fe. Water well drilled by the N.M.S.H.T.D. - Geology & Foundation Exploration Unit here, used to irrigate the trees, was drilled to a depth of 225 ft. with static water level at 130 ft.
4.2	226.5	Merge left into I-25 traffic.
4.9	227.2	On the right. The Sandia Mountains (Spanish

for "watermelon"), rise to an elevation of 10,678 ft. These Precambrian granite and metamorphic fault block mountains lie within the Sandia Uplift. I-25 at this point has an elevation of approximately 5,000 ft. Water wells for the city of Albuquerque located between here and the Sandias are from 984 to 1247 ft. in depth with water levels of around 328 ft.
The next 11 miles will be on Quaternary Intermediate Pediment deposits.

7.9	230.2	On the left. Approximately 62 miles to the west, Mount Taylor can be seen, a strato volcano with an elevation of 11,301 ft.
8.9	231.2	On the right is the N.M.S.H.T.D. - District 3 Headquarters.
11.4	233.7	Interchange to the Sandia Tramway, a cable car ride that rises from 6539 ft. to 10378 ft. in a distance of 2.7 miles.
11.6	233.9	Entering the Sandia Indian Reservation.
12.7	235.0	Entering Sandoval County, leaving Bernalillo County. On the left. Rio Rancho, on other side of the Rio Grande, elevation of 5290 ft. Water wells are approximately 984 ft. in depth with water levels at approximately 700 ft.
16.0	238.3	The next 7 miles lies on Quaternary Terrace deposits and Alluvial aprons.
17.7	240.0	On the left, located in the valley is the town of Bernalillo. Elevation of 5050 ft.
18.8	241.1	Leaving the Sandia Indian Reservation.
19.4	241.7	On the right. Red sandstone of the Santa Fe Formation seen in road cut.
19.6	241.9	Interchange construction.
20.6	242.9	The roadway for the next 2.3 miles exhibits frequent swales and sags, due to the presence of collapsible soils.
21.3	243.6	Entering the Santa Ana Indian Reservation.
22.8	244.1	To the left. The unique volcanic feature across the river is a diatreme (a breccia filled pipe that was formed by a gaseous

		explosion).
23.8	245.1	The next 19.8 miles lies on Quaternary-age Santa Fe Formation deposites.
23.9	245.2	The end of this collapsible soils area.
24.0	245.3	On the left and right. Albuquerque Materials Inc., Santa Ana Aggregate Plant.
25.5	246.8	Leaving the Santa Ana Indian Reservation.
25.8	247.1	On the right. Aggregate Specialist of New Mexico.
25.9	247.2	The roadway for the next 3.2 miles, at one time exhibited the same kind of riding surface as the last stretch. This area of collapsible soils was treated by dynamic impact methods, which was found to be the most cost effective.
26.5	247.8	Exit 248, Algodones (Spanish for "cotton"). The grade separation bridge structure ahead was designed by the N.M.S.H.T.D. - Bridge Section, and received an "Award of Merit" by the American Institute of Steel Construction, Inc. in 1984.
27.3	248.6	Entering the San Felipe Indian Reservation.
29.1	250.4	End of collapsible soils section.
30.2	251.5	Excellent quality gravel on both sides of highway (L.A. wear of 24.8 & soundness of 8.7). Note yellow compacted sand.
31.7	253.0	On the right in the distance. Gypsum, which occurs in the Jurassic-age Todilto Formation, is mined for use in the manufacture of wallboard and plaster.
32.3	253.6	Ahead in distance, lies the Southern Sangre De Cristo Mountains (Spanish for "Blood of Christ") above Santa Fe, with an elevation of 12,622 ft. The mountains are composed of Precambrian Granite.
33.9	255.2	On the right in the distance. The Ortiz Mountains, an intrusive complex. (More about these on the return trip).
34.5	255.8	On the left in distance. The Jemez Mountains, elevation at Redondo Peak is 11,254 ft. The

		peak is a structural dome in the center of the Valle Grande (Jemez) Caldera, which resulted from a very large volcanic explosion, leaving a caldera floor with a diameter of approximately 20 miles.
35.9	256.2	Leaving the San Felipe Indian Reservation. Two water wells have been drilled by the Geology & Foundation Exploration Unit slated for rest area usage. The well on the left was drilled to a 722 ft. depth, with water level at 357 ft., and producing approximately 125 gpm. The well on the right was drilled to a 587 ft. depth, with water level at 358 ft.
37.8	258.1	Entering the Santo Domingo Indian Reservation.
40.4	260.7	Ahead. Basalt capped La Bajada Mesa (Spanish for "descent").
40.9	261.2	Another view of the Ortiz Mountains on the right.
42.3	262.6	On the right, in the distance, is another gypsum mine. Mica is mined near here, as well.
42.8	263.1	Entering Santa Fe County, leaving Sandoval County. Entering into the Southern Rocky Mountain Physiographic Province. Entering into the N.M.S.H.T.D. - District 5's district.
43.0	263.3	Crossing over main line of the Atchison, Topeka, and the Santa Fe. The benched cut on the railroad to the right, is in the Cretaceous-age Dakota Sandstone.
44.1	264.4	Leaving the Santo Domingo Indian Reservation. Interchange to Cochiti Pueblo and Lake. Crossing a major fault, the red beds to the left are Tertiary Galisteo Formation sandstone.
44.8	265.1	On the left. Road cut is in red Espinaso Volcanics (breccia, conglomerate, sandstone, tuff, and various volcanic debris).
45.0	265.3	Crossing over another major fault, on the right in road cut the Cretaceous Mancos shale can be seen.
45.5	265.8	On the right. 6 ft. dark grayish-green

		diadase dike.
45.7	266.0	Climbing through the black basalt flow which caps La Majada Mesa (Spanish for "sheep-fold").
47.0	267.3	On the left, in the distance, is Tetilla Peak (a cinder cone with an elevation of 7206 ft.). On the right. The Cerrillos Hills (Spanish for "hills") a laccolith. Located on this side of the Cerrillos Hills are believed to exist the oldest turquoise mines in the United States, which were mined by pre-Spanish Indians.
47.6	267.9	The elevation at this point is approximately 6100 ft.
48.0	268.3	Santa Fe, NM ahead. Elevation of 7045 ft. Behind Santa Fe lies the Sangre de Cristo Mountains (Spanish for "Blood of Christ"), which are faulted blocks of Precambrian granite rising to an elevation of 12,622 ft. on Santa Fe Baldy and 13,102 ft. on Truchas Peak. The Sangre de Cristo's mark the southern limits of the Rocky Mountains.
50.0	270.3	On the left. Water is piped back up the hill to the Rest Area behind us from this well. Static water level is at 10.5 ft.
50.5	270.8	Left and right. Several monzonite intrusives of Tertiary-age.
53.4	273.7	On the right near water storage tank. The New Mexico State Penitentiary.
55.1	275.4	New construction on the Santa Fe by-pass to link up with U.S. 84/285.
56.9	277.2	Take Exit - 278. Proceed straight ahead. The return trip will bring us back to here. The road to the right is N.M. 14, which returns to Albuquerque, via several mining communities.
57.1		For those of you that thought that the dinosaurs roamed the area 65 to 225 million years ago, take a look to the right, in front of that house, and you will be able to observe Stegosaurus and Pteranodon.
57.7		Merge with Cerrillos Road / N.M. 14 traffic.
58.7		On the left. N.M.S.H.T.D. - District 3

Headquarters, Patrol/Maintenance Yard, the Field Exploration & Testing Section (where all but one of our illustrious geologists reside, on the few occasions that they are not out and about in the field), and the N.M. State Police Complex.

58.8 Santa Fe City Limits.

60.7 On the right. A sample of the unique southwestern art found here in Santa Fe, and in New Mexico, as a whole.

62.2 The Scenic Historic Marker reads: " Santa Fe - On the Camino Real - Pop. 48,899 Elev. 7045 Santa Fe, the oldest capital city in the United States, was established in 1610 as the seat of Spanish colonial government for the Province of New Mexico. The Palace of the Governors, used by Spanish, Mexican, and Territorial governors, has flanked the historic plaza since its construction in 1610, and now comprises part of the Museum of New Mexico. "

62.6 St. Michael's Drive to the right, goes by the College of Santa Fe Campus.

63.7 On the left. The U.S.P.H.S. Indian Hospital.

63.8 On the left. The Indian School Complex.

64.4 On the right. The N.M.S.H.T.D. - General Office Headquarters. The smaller building being the Materials & Testing Laboratory (where one of the other topnotch geologist resides. Back behind and across the branch line of the A.T. & S.F. railroad, lies the South Capitol Complex.

64.7 On the left. The New Mexico School for the Deaf.

64.9 Signal light. We will be turning left onto St. Francis Drive / U.S. 84/285. Approximately 1.0 miles straight ahead on Cerrillos Road is the State Capitol Complex and the Santa Fe Plaza.

65.6 Crossing over the Santa Fe River.

67.1 On the right. The National Cemetery.

67.5 Crossing over the divide between the Santa Fe River and the Tesuque Creek drainages. The elevation at the summit is 7332 ft.

67.9		An excellent view of the Sangre de Cristo Mountains.
71.5	170.5	On the left. Just past the Santa Fe Opera is the site of an old gravel pit.
74.7	173.7	On the left. The Tesuque Indian Pueblo (Spanish for "spotted dry place"), across the Tesuque Creek. The name is due to the intermittent flow of the creek.
75.2	174.2	Entering the Espanola Barrancas or Badlands. The formations are composed of volcanic ash and clay derived from the volcanic ash, which were deposited from volcanic erruptions of the Jemez Mountains. The badland formations owe their existence to the nature of the clays, to the arid climate, and to the lack of vegetation.
75.8	174.8	Ahead on the left. Camel Rock has been sculptured by nature through differential erosion of the siltstones, of the Santa Fe Formation.
77.8	176.8	Ahead, more badlands and the Jemez Mountains.
78.0	177.0	The village of Cuyamungue (derived from the Tewa Indian language, meaning "place where the Spanish live near").
78.4	177.4	On the right. The Cuyamungue Patrol Yard.
80.1	179.1	The village of Pojoaque (Spanish for "drink water place") was an Indian Pueblo until 1900.
81.2	180.2	We will be continuing on U.S. 84/285. N.M. 502, to the left, goes to Los Alamos and the Los Alamos National Scientific Laboratory.
88.9	187.9	The town of Espanola lies on Quaternary alluvium. Elevation of 5590 ft.
89.7	0.0	Continue on N.M. 68. U.S. 84/285 to the left.
92.3	2.6	On the right. Between the highway and the hills occur collapsible soils.
93.5	3.8	On the left. U.S. 285 to Ojo Caliente (Spanish for "hot eye"). Located approximately 21 miles from here Ojo Caliente is known for its hot baths. Defining the west side of the Rio Grande Rift,

hot (98 - 113 degrees Fahrenheit) mineral springs come to the surface in a faulted area where Precambrian granite and quartzite have been lifted above the Santa Fe Formation. The springs contain minor amounts of iron, arsenic, soda, and lithium.

93.9 4.2 On the right. The Scenic Historic marker reads: " San Gabriel. On the Camino Real. Governor Juan de Onate set up his headquarters in San Juan Pueblo in 1598, but by 1601 he had moved the Spanish Capital across the Rio Grande to Yuque Yunque Pueblo. Named San Gabriel, it served as the seat of government until 1610, when Onate's successor founded a new capital at Santa Fe. "

94.2 4.5 The basalt cap of Black Mesa ahead displays closely-spaced columnar jointing.
Chronology:
2.8 million years ago - Black Mesa's basalt cap. Between 2.8 and 2.4 million years ago - the development of the through-flowing Rio Grande. 2.4 million years ago - basalt from the Jemez Mountains flowed across the river gravels. 1 million years ago - the Jemez Caldera/Valle Grande exploded.

97.1 7.4 On the right. The Alcalde Patrol Yard. The water well here was drilled to 103 ft. with the static water level at 84 ft. The back side of the building has settled, with the southwest corner having settled 19 inches thus far, due to the presence of collapsible soils. Foundation investigation performed by the Geology & Foundation Exploration Unit, revealed SPT N-values of the following:

N-value	Depth (ft.)
5	5
10	7
6	10
7	15
19	20
6	22
7	25
14	40

99.6 9.9 On the right. Quaternary alluvial fan deposits.

102.1 12.4 On the right. Sand and gravel pit.

103.6 13.9 On the right. A gravel pit.

103.7	14.0	The village of Velarde lies on Quaternary alluvium.
104.9	15.2	Entering into the scenic Toas volcanic field of the Taos Plateau, which holds the Rio Grande in a narrow gorge. The road is built on alluvium and the Santa Fe Formation, with landslide debris above and tholeiitic-basalt flows of the Servilleta Basalt above that.
107.1	17.4	On the left. The national historic landmark of the American Society of Civil Engineers, commemorates the establishment in 1889 of the first river-gauging station by the U.S. Geological Survey.
107.3	17.6	On the left, across the river, can be seen Embudo Station. We will be returning here for lunch.
108.3	18.6	On the right. This area is a major rock fall area and a 10 ft. high wire rope net rock fence has been designed to be placed at the top of the cut slope with a wire rope net draped over the cut slope itself.
109.1	19.4	On the right. Cut is to be moved back 20 ft. and flattened.
109.4	19.7	Thru-cut is to be moved back 20 ft. and flattened.
110.2	20.5	N.M. 75 to the right. 8.3 miles up N.M. 75 is the entrance to the Harding mine. The mine was developed in a Precambrian pegmatite that contains large amounts of lepidolite, spodumene, tantalite-columbite, microlite, and beryl. The mine was last actively mined in 1958.
110.4	20.7	On the left. Santa Fe Formation crossbedded sandstone cliffs of uncertain age.
111.5	21.8	Same treatment as the last thru cut.
112.6	22.9	On the right. The water well at the Rinconada Patrol Yard is 69 ft. in depth with the static water level at 50 ft.
113.0	23.3	Same treatment as above.
113.4	23.7	Same treatment as above.
113.8	24.1	The Taos/Rio Arriba county line. We will be

ending our northeastward passage here and turning around.
Stop # 1. Exit from the busses for a discussion of the rock fall problems and the design recommendations for the Embudo project. On N.M. 68 at M.P. 25.0, a wire rope net will be draped over the cut slope face; and at M.P. 26.0 and M.P. 26.6, there will be a 10 ft. high wire rope net rock fence placed on the rock talus backslope.

The mountain side to the northeast is composed of Precambrian pink, gray, and green schist and pink and gray Ortega Quartzite. Abundant garnet and staurolite crystals occur in the schist.

Return to busses and retrace route to Embudo Station for lunch.

120.3	17.6	Turn right crossing bridge over the Rio Grande. Stop # 2. Embudo Station. Formerly a station on the Chili Line, a narrow-gauge railroad of the Denver and Rio Grande Railroad. This line went to Santa Fe, and was constructed in 1880 and abandoned in 1941.

" Let's Eat "

After lunch return to busses for the return trip that will retrace the route to Santa Fe.
0.2

162.9		Intersection of St. Francis Drive and Cerrillos Road, in Santa Fe. Turn right.
170.4		Retrace the route down Cerrillos Road traveling under I-25 and turning to the left onto the East Frontage Road.
170.5		Turn right into the Brugg Cable Products, Inc. drive. 0.2

Stop # 3. Exit busses for a tour of the Brugg Cable Products, Inc. assembly plant.

Return to busses and retrace route to N.M. 14 / Turquoise Trail.
0.3

171.0	46.2	The roadway for the next 8.2 miles lies on the Santa Fe Formation.

173.4	43.8	On the right. A cinder cone with the crater opening toward us.
174.0	43.2	On the right. The New Mexico State Penitentiary.
176.4	40.8	On the right. The Cerrillos Hills, a complex of intermediate sills, laccoliths, and plugs that have intruded into the Mancos shale, Mesaverde Formation, Galisteo sandstone, and volcanic breccias. Base metals and gem turquoise have been mined from these hills.
178.2	39.0	Ahead on the side of the Ortiz Mountains can be seen the mine dumps (on the right) and the spent ore deposits (on the left) from the Gold Field Corp. open pit gold mine. Approximately 250,000 ounces of gold was mined from 1979 to its closing in 1986.
179.6	37.6	End of new pavement.
180.2	37.0	Arroyo San Marcos. The Espinasa Formation is exposed in the sides and bottom of the arroyo.
180.3	36.9	The Santa Fe Formation is exposed in the cut.
181.3	35.9	Arroyo Coyote. Water discharges from the Santa Fe Formation.
182.5	34.7	The roadway crosses over an unconformable contact between the Santa Fe Formation and the Galisteo sandstone. The Galisteo strata are nearly vertical here. This is the southern margin of the Santa Fe Plateau and the northern margin of the Galisteo Basin.
182.8	34.4	The Galisteo Formation consists of approximately 4500 ft. of buff to reddish-brown conglomeratic sandstone, interbedded with red to brown siltstone, and shale. The Official Scenic Marker reads: " Garden of the Gods. Vertical beds of colorful sandstone and mudstone of the Galisteo Formation were deposited in streams 70 million years ago. Deposited as horizontal sheets, they have been tilted to their present vertical position by mountain building forces beneath the earth's surface. Elevation 6,000 ft. "
184.0	33.2	Steeply dipping shale, siltstone, and conglomeratic sandstone beds of the Galisteo Formation are exposed in the cut slope.

184.5	32.7	Crossing contact between the Mesaverde Formation and the Galisteo Formation. On the right, the Galisteo Formation rests unconformably on approximately 1000 ft. of the Mesaverde Formation.
184.6	32.6	The Mesaverde Formation is exposed in the cut slope.
185.0	32.2	Crossing over the contact between the Mesaverde Formation and the underlying Mancos shale. The Mancos shale, is a marine, dark bluish-gray, silty and calcareous shale, with several thin beds of limestone, and is approximately 2300 ft. thick.
185.2	32.0	Straight ahead. Monzonite laccolith of the Cerrillos Hills intruding the Mancos shale.
185.5	31.7	Bridge over the A.T. & S.F. Railroad. On the right, note the contorted and metamorphosed Mancos shale on the flanks of the Cerrillos Hills laccolith.
185.6	31.6	On the right. The Official Scenic Historic Marker reads: " Cerrillos. Elevation 5688 ft. Before the arrival of the Spanish, the mineral-rich area around Cerrillos produced turquoise which was traded as far as the Valley of Mexico. An early settlement of Los Cerrillos harbored Spanish refugees from the 1680 Revolt, but the present community was not founded until the lead strike of 1879. "
185.7	31.5	Crossing over Galisteo Creek.
186.1	31.1	County road 55 to the left was the road used to reach the Gold Field Corp. gold mine. On the right, the steeply dipping and contorted Mancos shale intruded by monzonite of the Cerrillos Hills laccolith, can be seen.
186.2	31.0	The roadway is on the Mesaverde Formation, which consists of about 1100 ft. of fine to medium grained sandstone, siltstone, shale, and bituminous and semi-anthracite coal beds. The dark red beds, to the left, are metamorphosed shale.
188.1	29.1	Entering Madrid. Note the dumps from the abandoned coal mines.
188.5	28.7	At 11:00 o'clock. Location where the tipple of the coal mine used to be. A monzonite sill

crops out above.

189.3	27.9	On the left. Dump of the Cerrillos-Madrid coal district, last active back around 1952.
189.5	27.7	On the right. Carbonaceous shale and coal exposed in the cut slopes.
190.4	26.8	At 9:00 - 11:00 o'clock. The Ortiz Mountains, consisting of a complex of intermediate sills, laccoliths, dikes, and plugs intruded into the Mancos shale, Galisteo Formation, and the Mesaverde Formation.
190.6	26.6	On the right. Monzonite intrusions in the cut slopes. Picturesque view back towards Santa Fe.
192.3	24.9	On the right. Gravel pit.
193.0	24.2	Ahead. Metamorphosed shale of the Mesaverde Formation overlain by monzonite.
193.2	24.0	Contorted shale of the Mesaverde Formation overlain by monzonite.
193.4	23.8	Metamorphosed Mesaverde Formation shale in cut slope.
193.6	23.6	Monzonite dike intruding the Mesaverde Formation.
194.5	22.7	Excellant view of the eastern backslope of the Sandia Mountains.
194.6	22.6	On the right. A monzonite intrusion in the cut slope.
194.8	22.4	On the right. Another monzonite intrusion in the cut slope.
195.1	22.1	The approximate southwestern margin of the Galisteo basin and the northeastern margin of the complex structural area of the Hagan basin and the Sandia Mountains.
195.8	21.4	At 9:00-11:00 o'clock, the Ortiz Mountains. At 11:30-12:30 o'clock, the San Pedro Mountains, a complex laccolith which intruded strata of Pennsylvanian-age rocks. The intrusion locally metamorphosed the shales into hornfels and the limestones into marble-like rock. There have been numerous copper, silver, and gold mines intermittently operated since prior to the

		coming of the Spaniards.
197.8	19.4	At 2:00-3:00 o'clock, the Hagan basin contains rocks ranging in age from the Pennsylvanian-age Sandia Formation to the Tertiary-age Santa Fe Formation.
198.7	18.5	On the right. Exposed in the bank of the Cuchillo Arroyo is red shale of the Chinle Formation. On the left. Contact between the Todilto limestone and gypsum and the underlying Entrada sandstone. The Todilto limestone consists of approximately 8 ft. of dark-gray, thinly-laminated, fetid limestone overlain by about 50 ft. of massive gypsum. The Entrada sandstone consists of a tangentially cross-bedded, flesh colored, pink, or buff, medium grained sandstone.
198.95	18.25	On the left. The entrance to the Pegasus Gold Corp. Lucus Canyon and Carache Canyon gold mines. These mines are in the exploration/evaluation phases. Both areas are associated with volcanic intrusions in the form of sills, dikes, and necks into the Mesaverde Formation. Crossing over a high angle thrust fault, the plate of which is thrust westward.
199.2	18.0	The hill on the left, is underlain by the Glorieta sandstone and the San Andres limestone.
199.25	17.95	In the center of the bench on the right lies a north-trending normal fault, downthrown to the east. Arkosic limestone of the Madera limestone is on the west side of the fault and limestone and Glorieta sandstone of the San Andres Formation are exposed in the cut slope.
199.55	17.65	Golden, N.M. Site of one of the first placer gold discoveries in the State.
199.9	17.3	On the right. Old foundations from the small mill for ores that were mined in the San Pedro Mountains.
200.1	17.1	On the right. Steep northwest dipping beds of arkosic limestone of the Madera limestone, in the arroyo bottom.
200.4	16.8	On the right. The saddle between the hills is crossed by a normal fault, which is downthrown to the north. The hill to the right is

		underlain by the arkosic limestone of the Madera limestone, and the hill to the left is underlain by Precambrian quartzite.
201.85	15.35	Thru cut is in the Madera limestone.
202.9	14.3	Begin new pavement. Leave Santa Fe County, enter Sandoval County. Re-enter N.M.S.H.T.D. - District 3 maintenance.
203.1	14.1	Precambrian quartzite in the hills on the left.
203.4	13.8	On the left. The brownish formation behind the quartzite ridge is Precambrian gneiss.
204.1	13.1	On the right in the arroyo, gneiss and quartzite resting on the Madera limestone. This is the southerly continuation of the thrust fault.
204.3	12.9	On the right. The Abo Formation is resting conformably on the Madera limestone. The Abo Formation consists of approximately 950 ft. of fine to coarse, buff, red, brown sandstone and arkose alternating with red shale and siltstone.
204.5	12.7	Road crosses over the thrust fault. The cuts are in the Abo Formation.
205.4	11.8	On the right in the arroyo. The sandstone and arkosic limestone of the Madera limestone.
206.4	10.8	Crossing over San Pedro Creek.
206.45	10.75	Site of the pit used for surfacing material for the stretch of road that we are on.
206.7	10.5	Leave Sandoval County, enter Bernalillo County.
207.7	9.5	On the right. An outcrop of arkosic sandstone in the Abo Formation.
207.85	9.35	On the right and left. The Paako State Monument Indian Ruins.
209.5	7.7	An excellant pinion nut picking area, when in season.
210.5	6.7	San Antonito, N.M.
211.3	5.9	N.M. 536 to Sandia Crest. The Sandia Mountains are part of the Sandia - Manzanita - Manzano - Los Pinos block faulted range. The western front of the range is underlain by Precambrian

		granite, schist, quartzite, and gneiss. The rocks are overlain unconformably by about 150 ft. of primarily Pennsylvanian-age Madera limestone, which contain brachiopods, bryozoans, corals, and crinoid stems. Strata of the mountains were involved in late Upper Cretaceous and early Tertiary thrusting.
211.8	5.4	On the right, in cut. The Upper Triassic Chinle Formation, a brick-brown mudstone with thin sandstone ledges. On the left, in the hill. The upper part is the Dakota and Morrison Formations with the Todilto and Entrada Formations below.
212.4	4.8	The Chinle Formation is exposed in the thru-cut. Crossing the divide between the Tijeras Creek and the San Pedro Creek drainages.
213.1	4.1	The Chinle Formation in the cut slope to the left and in Arroyo San Antonio to the right. At 12:30 o'clock the Abo crops out in a steep east-dipping monocline (about 2 miles from the road). This is the eastern boundary of the Sandia Mountains.
213.3	3.9	On the left. Exposed in the hill, is the Todilto limestone and the Wanakah gypsum.
213.4	3.8	On the left. From the arroyo bottom upward, lie the Chinle Formation, the Entrada sandstone, and the Todilto Formation.
213.5	3.7	Cedar Crest, N.M. City Limits.
214.0	3.2	On the left, in the cut slope. Steeply dipping and highly contorted strata of the Morrison Formation. The Tijeras Canyon fault lies just behind the hill.
214.2	3.0	At 1:30 o'clock, in the distance, the Ideal Cement Co. - Portland Cement Plant, can be seen.
214.4	2.8	On the left. On the hill side are the Mesaverde and Mancos shale Formations. The Tijeras Canyon fault, which is downthrown to the east, is in the bottom of the arroyo. N.M.14 is on the Chinle Formation. The Dakota sandstone, Morrison Formation, and Entrada sandstone, are cut out by the fault.
214.5	2.7	Crossing over the Tijeras Canyon fault trace.
214.7	2.5	On the left. Contorted strata of the Mesaverde

		and Mancos shale Formations.
215.0	2.2	We are in a strike valley formed by the erosion of the Mancos shale.
215.8	1.4	On the right. Approximately 100 ft. of Dakota sandstone is exposed in the hill side. The Dakota sandstone consists of buff, fine to conglomeratic, thin to massive-bedded sandstone interbedded with gray shale.
216.1	1.1	On the left. Nearly vertical beds of the Mesaverde Formation on the hill.
216.4	0.8	On the right, in the cut slope, outcrops the Dakota sandstone.
216.7	0.5	On the right, in the cut slope, outcrops the Mesaverde Formation.
216.9	0.3	Crossing the Gutierrez fault, a northeast trending fault, downthrown to the northwest, between the Mesaverde Formation (to the northwest) and the Abo Formation (to the southeast).
217.0	0.2	On the left and right. Sandstone and shale beds of the Abo Formation. Tijeras Village Limits (Spanish for "scissors"). Elevation 6300 ft.
217.1		Turn right to head west on I-40. On the right. Note the gabion and the 4 ft. rockfall fence.
217.5	174.45	On the left and right. The Abo Formation is exposed in the thru-cut.
217.7	174.25	On the right. The cut slope shows the contact between the Abo Formation and colluvium derived from debris of the arkosic member of the Madera limestone.
217.8	174.15	On the right. Another rockfall fence.
217.9	174.05	On the right. Red colored shotcrete over wire mesh drape.
218.0	173.95	On the left. The Ideal Cement Co. is quarrying in the Madera limestone.
218.3	173.65	At 11:00 o'clock, a small fault can be seen, in the Madera limestone, downthrown to the north.

218.4	173.55	On the right. The cut slope is in the lower member of the Madera limestone. The member is about 470 ft. thick in this vicinity. Note the shotcrete over wire mesh drape.
218.7	173.25	On the right. The cut slope consists of colluvium over the arkosic member of the Madera limestone. A rockfall fence has been placed at the toe of the slope.
218.8	173.15	On the right. The cut slope consists of colluvium. The continuation of the rockfall fence.
218.85	173.1	On the left and right. Colluvium lies uncomformably on Precambrian greenstone. The greenstone is a foliated hornblende and chlorite metavolcanic sequence, apparently derived from andesitic and basaltic flows and some intrusive plugs and sills.
219.0	172.95	On the right. The cut slope has been laid back to a 1-3/4:1 slope angle into colluvium over greenstone. Crossing over the trace of the Tijeras Canyon fault.
219.3	172.65	On the right. A 6 ft. gabion has been placed at the toe of Precambrian greenstone (note the sign on the gabion that reads "Pet Rock Prison"). Colluvium rests on both sides and over the greenstone.
219.4	172.55	On the right. Exposed in the cut slope is Precambrian brownish-pink gneiss.
219.7	172.25	On the right. A lamprophyre dike in Precambrian gneiss.
219.75	172.20	On the right. A wire mesh drape has been placed over Precambrian gneiss.
220.1	171.85	On the right. Another wire mesh drape has been placed over Precambrian gneiss with several lamprophyre dikes.
220.4	171.55	On the left. A wire mesh drape and a rockfall fence have been placed. From left to right, across the cut slope, brownish-pink gneiss, purplish-gray gneiss, purplish-gray quartzite, and brown gneiss are exposed.
220.5	171.45	On the right. The cut slopes across Tijeras

		Creek are in brown gneiss.
220.7	171.25	On the left. Contact between the Precambrian gneiss and the Precambrian granite.
220.9	171.05	On the left. The cut slope is in granite with a 2-1/2 ft. lamprophyre dike in the middle of the cut.
221.0	170.95	Crossing over Tijeras Creek.
221.7	170.25	Left and right. On the hillsides can be seen typical weathering exposure of the granite.
222.8	169.15	At 2:00 o'clock, the smooth shear plane was the site of a major rockfall in 1985. Granite fell onto the roadway colliding with a pickup truck. Rockbolts have been installed.
223.5	168.45	On the right. Another wire mesh drape.
223.75	168.20	On the right. The dark yellowish-brown granite, is in the shear zone of the Sandia Mountains.
224.25	167.70	Crossing the approximate trace of the frontal block fault of the Sandia Mountains.
228.9	163.05	Take Exit 162 to Louisiana Blvd.
229.2		Turn right at yield sign.
230.0		Turn right into parking lot of the Ramada Hotel Classic.

END of DAYS TRIP

Road Log References

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41st ANNUAL HIGHWAY GEOLOGY SYMPOSIUM
RAMADA HOTEL CLASSIC
ALBUQUERQUE, NEW MEXICO
August 15-17, 1990

LIST OF ATTENDEES

Ralph L. Abeyta
Testing Laboratories, Inc.
P. O. Box 1144
Alamogordo, NM 88311

A. Kulathu Aiyer
Dept. of Civil Engn., NMSU
Box 30001, Dept. 3CE
Las Cruces, NM 88003

Yinka Akinyemi
Hilfiker Texas Corporation
637 West Hurst Blvd.
Hurst, TX 76053

Jamie Barnes
Sergent, Hauskins & Beckwith
2810 Chico Lane
El Paso, TX 79903

William D. Bingham
Law Engineering
3301 Atlantic Avenue
Raleigh, NC 27619

Vernon L. Bump
South Dakota DOT
221 West 4th
Pierre, SD 57501

Chien-Tan Chang
Federal Highway Administration
6300 Georgetown Pike
McLean, VA 22101

Koon Meng Chua
University of New Mexico
Dept. of Civil Engineering
Albuquerque, NM 87131

Russell E. Clemons
NM State University
Dept. of Earth Sciences
Las Cruces, NM 88003

Richard H. Cross
New York State Thruway Authority
200 Southern Blvd.
Albany, NY 12201

John Duffy
California DOT
5900 Folsom Blvd.
Sacramento, CA 95819

Gordon M. Elliott
Golder Associates, Inc.
3730 Chamblee Tucker Rd.
Atlanta, GA 30341

Kelley V. Fetter
Contech Const. Products, Inc.
150 Woodward Road, S.E.
Albuquerque, NM 87102

David P. Findley
Golder Associates, Inc.
4104 148th Avenue, N.E.
Redmond, WA 98052

Joseph J. Finney
Dept. of Civil Engn., NMSU
Box 30001, Dept. 3CE
Las Cruces, NM 88003

Susan Gallaher
New Mexico State Hwy & Trans. Dept
P. O. Box 1149
Santa Fe, NM 87504

Joseph A. Gutierrez
Vulcan Materials Co.
P. O. Box 4195
Winston Salem, NC 27104

William C. Haneberg
NM Bureau of Mines
Campus Station
Socorro, NM 87001

Robert Hatcher
Central Mine Equipment
6200 North Broadway
St. Louis, MO 63147

Richard Heibel
New Mexico State Hwy & Trans. Dept
P. O. Box 1149
Santa Fe, NM 87504

Scott I. Hensley
Contech Const. Products, Inc.
150 Woodward Road, S.E.
Albuquerque, NM 87102

Steve Huddleson
N.M. State Highway Dept.
P.O. Box 3768, Station D
Albuquerque, NM 87190

Richard Humphries
Golder Associates, Inc.
3730 Chamblee Tucker Rd.
Atlanta, GA 30341

Charles T. Janik
Penn. DOT
1118 State Street
Harrisburg, PA 17120

Brian P. Jones
Contech Const. Products, Inc.
150 Woodward Road, S. E.
Albuquerque, NM 87102

G. Bruce Kay
Arizona DOT
206 S. 17th Avenue
Phoenix, AZ 85007

Kathryn Kretz
N.M. State Highway Dept.
P.O. Box 3768
Albuquerque, NM 87190

Clyde Langdale
New Mexico State Hwy & Trans. Dept
P. O. Box 1149
Santa Fe, NM 87504

Alain J. Lazard
Diversified Ski Services
P. O. Box 2006 (1429 Lanny Lane)
Olympic Valley, CA 95730

Nolan LeBlanc
New Mexico State Hwy & Trans. Dept
P. O. Box 1149
Santa Fe, NM 87504

Leo A. Legatski
Elastizell Corp. of America
P. O. Box 1462
Ann Arbor, MI 48106

Richard Lueck
New Mexico State Hwy & Trans. Dept
P. O. Box 1149
Santa Fe, NM 87504

Willard McCasland
Oklahoma DOT
200 N. E. 21st Street
Oklahoma City, OK 73105

Harry L. Moore
Tennessee DOT
P. O. Box 58
Knoxville, TN 37901

Andres Munoz
Federal Highway Admin.
819 Taylor Street
Fort Worth, TX 76102

Thomas C. Neel
The Neel Company
6520 Deepford Street
Springfield, VA 22150

Peter J. Nicholson
Nicholson Const. of America
P. O. Box 308
Bridgeville, PA 15017

Ed Rector
New Mexico State Hwy & Trans. Dept
P. O. Box 1149
Santa Fe, NM 87504

Eric B. Rehwoldt
GeoCon, Inc.
6960 Flanders Drive
San Diego, CA 92121

Gary Riedl
Wyoming Highway Dept.
P. O. Box 1708
Cheyenne, WY 82002

Kenneth H. Rippere
Golder Associates
3730 Chamblee Tucker Road
Atlanta, GA 30068

Dave Robertson
Hilfiker Texas Corporation
637 West Hurst Blvd.
Hurst, TX 76053

Larry Rockers
Kansas DOT
2300 Van Buren
Topeka, KS 66047

Joseph Romero
New Mexico State Hwy & Trans. Dept
P. O. Box 1149
Santa Fe, NM 87504

Mike Schulte
Wyoming Highway Dept.
Box 1708
Cheyenne, WY 82002

S. A. Senior
Ministry of Trans., Ontario
1201 Wilson Avenue, Downsview
Canada M3M 1J8,

Willard L. Sitz
Alabama Highway Department
1409 Coliseum Blvd.
Montgomery, AL 36130

Ernest L. Smith
Western Area Power Admin.
P. O. Box 3402
Golden, CO 80401

Gilbert Sparkman
New Mexico State Hwy & Trans. Dept
P. O. Box 1149
Santa Fe, NM 87504

Irving G. Studebaker
Montana Tech
Mining - W. Park
Butte, MT 59701

Robert A. Thommen, Jr.
Brugg Cable Products, Inc.
RR. 16, Box 198-E, 11 Frontage Rd.
Santa Fe, NM 87505

Sam I. Thornton
Univ. of Arkansas
BEC 4190
Fayetteville, AR 72701

Edwin Varela
New Mexico State Hwy & Trans. Dept
P. O. Box 1149
Santa Fe, NM 87504

Kenneth R. White
Dept. of Civil Engn., NMSU
Box 30001, Dept. 3CE
Las Cruces, NM 88003

Burrell S. Whitlow
Geotechnics
321 Walnut Ave. - P. O. Box 217
Vinton, VA 24179

William A. Wisner, Jr.
Florida DOT
P.O. Box 1029
Gainesville, FL 32602

Earl Wright
Kentucky Dept. of Highways
700 Wilkinson Blvd.
Frankfort, KY 40622

James B. Yarbrough
NMSH & TD
P. O. Box 1149
Santa Fe, NM 87504