45 TH

HIGHWAY GEOLOGY SYMPOSIUM

PORTLAND, OREGON
August 17-19, 1994

PROCEEDINGS

Scott F. Burns, Editor

Sponsored by
Portland State University
Oregon Department of Transportation
Washington Department of Transportation
Transportation Research Board

45TH HIGHWAY GEOLOGY SYMPOSIUM

PORTLAND, OREGON
AUGUST 16-20, 1994

LOCAL COORDINATING COMMITTEE

Scott Burns, Dept. of Geology, Portland State Univ.

Harry Ludowise, Retired FHWA, Vancouver, WA

Sue D'Agnese, Oregon DOT

Bob Van Vickle, Oregon DOT

Steve Lowell, Washington DOT

SPONSORED BY:

Portland State University

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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
1991	Albany, NY	1992	Fayetteville, AR

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. j 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 relativity 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 with 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 Francisco Bay area which illustrated the planning, construction and maintenance of transportation systems. In 1985, the one day trip illustrated new highway construction procedures int he 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.

* Revised from the 41st Highway Geology Symposium Proceedings.

(Taken from the 43d Highway Geology Symposium Proceedings)

EXHIBITORS

You are invited to visit their displays in the Mt. St. Helens Room. They are listed by booth numbers:

1) Hilfiker Retaining Walls, Eureka, California

2) A.B. Chance Co., Centralia, Missouri

3) Maccaferri Gabions Inc., Seattle, Washington

4) KeyStone Pacific Northwest, Beaverton, Oregon

5) Hayward Baker Inc., Seattle, Washington

6) R.S.T. Instruments Inc., Yakima, Washington

7) Tensar Earth Technology Inc., Atlanta, Georgia

8) Preformed Line Products, Cleveland, Ohio

9) Geokon Inc, Lebanon, New Hampshire

10) Brugg Cable Products Inc, Santa Fe, New Mexico

11) Piling Accessories, Matthews, North Carolina

12) Camp Dresser McKee, Albuquerque, New Mexico

13) Central Mining Equipment, Earth City, Missouri

14) Jensen Drilling Co., Eugene, Oregon

15) Surface Systems Inc., Truckee, California

16) Slope Indicator Co., Seattle, Washington

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Highway Geology Symposium

EMERITUS MEMBERS OF THE STEERING COMMITTEE FOR THE HIGHWAY GEOLOGY SYMPOSIUM

R.F. Baker

Virgil E. Burgat

Robert G. Charboneau

Hugh Chase

A.C. Dodson

Walter F. Fredericksen

John Lemish

George S. Meadors, Jr.

David Mitchell

W.T. Parrot

Paul Price

David L. Royster

Mitchell Smith

Ed J. Zeigler

Berke Thompson

Bill Sherman

Burrell Whitlow

Highway Geology Symposium

MEDALLION AWARD WINNERS

Hugh Chase	1970
Tom Parrot	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
Joseph A. Gutierrez	1990
Willard McCasland	1990
W.A. "Bill" Wisner	1991
David Mitchell	1993

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.

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AUGUST 16-20, 1994

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List is complete as of August 15, 1994

45th Highway Geology Symposium
August 17-19, 1994
Shilo Inn at the Airport
Portland, Oregon

Conference Program

Tuesday, August 16, 1994

8:30 AM to 5:30 PM Optional Field Trip to Oregon Coast, Leave from Main Lobby, Shilo Inn

6:00 PM to 9:00 PM Registration in Shilo Inn Main Lobby

Wednesday, August 17, 1994

8:00 AM to 5:00 PM Registration, Shilo Inn Conference Center

8:00 AM to 6:00 PM **Exhibitor Display**, Mt. St. Helens Room

9:00 AM **Technical Session I**, Mt. Rainier Room

Welcoming Remarks:
Dr. Lindsay Desrochers, Vice President for Finance and Administration, Portland State University
Member of HGS Steering Committee
Bob Van Vickle, Oregon DOT,
Steve Lowell, Washington DOT

9:20 AM Keynote Address: "The dynamic geology of Oregon"
Scott Burns, Portland State University

9:50 AM Coffee break with juices and Danish rolls, Exhibit Hall, Mt. St. Helens Room

COFFEE BREAK SPONSORED BY GEOKON, INC.

Technical Session II, Slope Failures & Stabilization, Mt. Rainier Room

Sue D'Agnese, Oregon DOT, Roseburg, OR, Moderator

10:20 AM Mark Koelling, Hayward Baker Inc., Seattle, WA

"Ground Improvement Case Histories for Highway Construction"

10:40 AM Robert E. Johnston, and Lee W. Abramson, Parsons Brinckerhoff, San Francisco, CA

"Geosynthetic Reinforcing of Highway Embankments"

11:00 AM **Joe Bailey,** Tensar Earth Technologies, Cambridge, MA

"Application of MSE (Mechanically Stabilized Earth) Slopes as Replacement for Retaining Walls; a New Hampshire Case History"

11:20 AM J.C. Kuhne and F.R. Glass, Geotechnical Unit, North Carolina Department of Transportation, Asheville, NC

"Probabilistic Analysis and Design of Slopes on Interstate 26 From Asheville, NC to Tennessee"

11:40 AM Lee R. Shaw, Chin Leong Toh, and Sam I Thornton, University of Arkansas, Fayetteville, Arkansas

"Effects of Lime on Cannon Creek Embankment Soil"

Noon: **Lunch:** Shilo Inn Executive Buffet, Mt. Hood Room, (roast beef, ham, turkey, corned beef and cheeses for sandwiches; potato salad, fresh fruit platter, relish tray, assorted breads, coffee, tea, decaf, iced tea - included with full registration)

1:30 PM **Technical Session III, Rockfall and Rock Excavation,** Mt. Rainier Room

Larry Pierson, Chief Engineering Geologist, Oregon DOT, Salem, Oregon, Moderator

1:30 PM Tom A. Armour, Mark S. Johnston, & Paul B. Groneck, Nicholson Construction Company, Roseville, CA

"Innovative Rock Anchoring at Boundry Dam, Washington"

1:50 PM Tom Badger, Washington State Department of Transportation, Geotechnical Branch

"Rock Excavations in Steep Ground"

2:10 PM Robert Flatland, Geological Engineering Division, University of Nevada, Reno, NV, Robert J. Watters, Geological Engineering Division, University of Nevada, Reno, NV, and David Cochrane, Nevada Department of Transportation, Carson City, NV

"Application of the Rockfall Hazard Rating System to Rock Cuts in Mountainous Terrain"

2:30 PM Robert A. Thommen, Brugg Cable Products, Inc., Santa Fe, NM, and Manuel A. Montejo, Cementos Apasco, NM

"Rock Retaining Wire Rope Net System Installation at Orizaba, Mexico"

2:50 PM **Refreshment Break**, Coffee, Tea, Soft Drinks, Exhibit Room, St. Helens Room

3:20 PM **Technical Session IV, Current Topics in Highway Geology**, Mt. Rainier Room, Larry Pierson, Moderator

3:20 PM Richard Hannan, and Michael Moran, Portland District U.S. Army Corps of Engineers, Portland, OR, David Scofield, Squier Associates, Lake Oswego, OR

"Design, Construction, and Performance of Horizontal Drains at Bonneville Navigation Lock, Oregon"

3:40 PM Rich Humphries, Golder Associates Inc., Atlanta, GA

"Recent Geotechnical Advances in the Design and Construction of Highway Tunnels"

4:00 PM Steven M. Huddleson, Camp Dresser & McKee Inc., Albuquerque, NM, and Anne Lovely, Esq., New Mexico State Highway and Transportation Dept., Albuquerque, NM

"Hazardous Materials in the Roadway"

4:20 PM S. Javed, Geotech Engineering and Testing, Houston, TX, C.W. Lovell, and D. A. Eastwood, School of Civil Engineering, Purdue University, W. Lafayette, IN

"Waste Foundry Sand in Subgrade and Controlled Low Strength Material"

4:40 PM Robert L. Burk and Norm Norrish Golder Associates, Redmond, WA and S.M. Lowell Washington DOT, Olympia, WA

"Investigation, Design, and Construction of the Spirit Lake Memorial Highway, Washington"

5:00 PM Vans start shuttling people to the Edgefield Inn for the Icebreaker party, leaving from Shilo Inn Main Lobby

Icebreaker Party at Edgefield Inn, Troutdale, OR

5:30 PM First tour of the grounds, brewery, and winery

6:15 PM Second tour of the grounds, brewery and winery

5:30 PM Beer and wine tasting in the Ballroom

7:00 PM Dinner: Sliced turkey breast, dressing, vegetable ragout, crisp vegetable tray and dips, fruit and cheese display, fresh market vegetables, pasta salad, green salad, coffee, tea, decaf, chef's selection of desserts & no host bar after all of the tasting is gone

8:00 PM First van back to hotel

11:30 PM Last van back to hotel

Thursday, August 18, 1994 Field Trip to St. Helens

7:30 AM leave from Lobby of Shilo Inn Convention Center

Stop 1: Mt. St. Helens National Volcanic Monument Visitors Center
Stop 2: Corps of Engineers Sediment Retention Structure
Stop 3: Hoffstadt Bluffs Viewpoint
Stop 4: Hoffstadt Creek Bridge Viewpoint
Stop 5: Elk Rock Viewpoint, Lunch

LUNCH SPONSORED BY BRUGG CABLE CABLE PRODUCTS INC.

DRINKS SPONSORED BY GOLDER ASSOCIATES

Stop 6: North Fork Viewpoint
Stop 7: Coldwater Lake
Stop 8: Spirit Lake Outlet Tunnel (depends on road)

6:00 PM return to Shilo Inn

6:00 PM **Highway Geology Symposium Steering Committee** Meeting, St. Helens Room (please check on this)

7:00 PM **Banquet Social Hour** begins, Shilo Inn, Mt. Hood Room

7:45 PM **Banquet:** The Holladay Buffet: Baron of beef, salmon, tossed green salad and dressings, potato salad, pasta salad with bay shrimp, Italian zucchini salad, relish tray, fresh fruit platter, warm rolls/butter, cheese mirror, assorted dessert table, coffee, tea, decaf, iced tea.

8:45 PM **Announcements:** Next years meeting (Ken Ashton of West Virginia), awards, and

Dinner speaker: Sue D'Agnese, Oregon DOT, slide show and talk on the "Wilson River Landslide"

Friday, August 19, 1994

8:00 AM **Technical Session V, Landslide Investigation and Stabilization,** Mt. Hood Room,

Keith Turner, Professor, Colorado School of Mines, Moderator

8:00 AM Jim Dahill, Wyoming Department of Transportation, Geology Branch, Cheyenne, WY

"Investigation and Stabilization of a Developing Landslide, Highway 28, South of Lander, Wyoming"

8:20 AM G. Michael Hager, and Mark Falk, Wyoming Department of Transportation, Cheyenne, WY

"Catastrophic Embankment Failure South of Sheridan, Wyoming, Interstate 90 - M.P. 41.1"

8:40 AM William F. Kane, University of the Pacific/Neil O. Anderson and Associates, Stockton, CA, and Timothy J. Beck, California Department of Transportation, Sacramento, CA

"Development of a Time Domain Reflectometry System to Monitor Landslide Activity"

9:00 AM Ricki M. Kobernik, Frank N. Toor, and Richard Watanabe, Oregon Department of Transportation, Roseburg, OR

"The Arizona Inn Landslide, Curry County, Oregon"

9:20 AM Gary L. Peterson, L. Radley Squier, and David H. Scofield, Squier Associates, Lake Oswego, OR

"Engineering Geology and Hydrology Update, Arizona Inn Landslide, Curry County, Oregon"

9:40 AM **Coffee Break**, Exhibit Room, Mt. St. Helens Room

10:20 AM **Technical Session VI, Current Topics in Highway Geology,** Mt. Rainier Room, Keith Turner, Moderator

10:20 AM D.W. Bruner, J.C. Choi, and T.R. West, Department of Earth & Atmospheric Sciences, Purdue University, West Lafayette, IN

"Evaluation of Indiana Aggregates for Use in Bituminous Highway Overlays"

10:40 AM Lanny H. Fisk, and Lee A. Spencer, F & F GeoResource Associates, Inc., Bend, OR

"Highway Construction Projects Have Legal Mandates Requiring Protection of Paleontologic Resources (Fossils)"

11:00 AM Donald V. Gaffney, Michael Baker Jr. Inc., Beaver, PA

"BV-116: The Bridge by Addendum: Wetlands Dictate a Change in Design"

11:20 AM R. Panneer Selvam, Robert P. Elliott, and A. Arounpradith, Department of Civil Engineering, University of Arkansas, Fayetteville, AR

"Pavement Analysis Using ARKPAV"

11:40 AM William F. Sherman, Consultant, Cheyenne, WY

"The Impact of Registration on Geologists on the Professional Involved in Highway Geology"

Noon **Official Closure of the 45th HGS Conference**

1:30 PM **Optional Field Trip up the Columbia Gorge**
Leave from Main Lobby of Shilo Inn

5:30 PM Return of the Columbia Gorge Field Trip

Saturday, August 20, 1994

"Workshop on the Engineering Geologist's Role in Wetland Identification, Management and Mitigation"
(With special emphasis on transportation industry issues)

Sponsored by
Committees A2L01 and A2L05
Transportation Research Board
Oregon Department of Transportation

8:30 - 9:00 AM Registration, Coffee and Rolls, Lobby, Shilo Inn Convention Center

8:55 AM Welcome, Jeff Keaton, AGRA, Salt Lake City & Chair of A2L05, Mt. Hood Room

9:00 AM : Mike Shippey, Wetland Resource Specialist, Oregon DOT

"Why is it a wetland? Characteristics and regulatory context"

9:30 AM William Fletcher, Wetland Hydrologist, Oregon DOT

"Wetland Water Quality Enhancement"

10:00 AM Coffee and Rolls, Lobby, Shilo Inn Convention Center

10:30 AM Jim Goudzwaard, Wetland Specialist, Army Corps of Engineers, Portland District

"Mitigation and Wetland Banks"

11:00 AM Tom Kuper, Principal and Project Geologist, David Newton and Associates, Portland, OR

"Design and Construction of a Wetland: Two Case Histories in Portland, Oregon"

11:30 AM Robert Tiedemann, Ecological Design, Boise, Idaho

"Use of Remote Sensing for Identification of Wetlands and Design of Mitigation"

Noon **Executive Buffet Lunch**, Mt. Rainier Room
(for menu, see Wednesday's lunch)

1:15 PM **Field Trip to Wetlands of the Portland Area:** Leaders: Mike Shippey, William Fletcher, and Craig Markham of Oregon DOT; Tom Kuper, David Newton and Associates.

a) **Columbia Corridor South Shore Area:** mitigation of two wetlands (one for ODOT and one for a business park) mentioned in Tom Kuper's talk.

b) **Trojan Nuclear Site Wetlands:** An ODOT site where a degraded wetland was enhanced for mitigation. This is an example of a qualified success and the problems that a mitigation site can encounter.

c) **Dalton Lake Mitigation Wetland Bank:** We will visit an area proposed by ODOT as a wetland mitigation bank and discuss its formation and management.

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Ground Improvement Case Histories For Highway Construction

Mark A. Koelling[1]

ABSTRACT

This paper presents the design and construction practice for various ground improvement methods applied to highway and bridge construction. The methods include the Vibro Systems (Vibro Compaction and Vibro Replacement), also Dynamic Deep Compaction, Compaction Grouting, Chemical Grouting, Jet Grouting, and Lime/Fly Ash Injection. An overview of each method is presented including a discussion of the mechanism of improvement (i.e. densification, reinforcement, or adhesion). The geotechnical considerations addressed by these methods include bearing capacity, settlement, slope stability, excavation support/underpinning, and groundwater control. Numerous case histories from throughout North America are presented to illustrate construction methods.

INTRODUCTION

The use of Ground Improvement technology (also known as Ground Modification or Stabilization) in highway construction in North America has increased significantly over the past 10 years. The earliest technology transfer to highway work of most methods is generally no older than 20 years. Several factors have impeded or slowed this process of implementation of these methods into geotechnical highway work. As discussed by Klinedinst and DiMaggio (FHWA) in a 1983 paper titled "Pitfalls in Implementing New Technology," the following list outlines some of the more prominent issues:

1. lack of continuity and coordination between design and construction phases of most contracting agencies;

2. insufficient subsurface investigations;

3. lack of practical, simplified design information;

4. clear assignment of risk and liability;

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5. specification type (performance or procedural);

6. contractor selection and qualification

7. quality assurance and control.

Many of these issues remain today, yet to be overcome by blending the resources and skills of the project construction teams, consisting of owner, consultant/designer, and contractor/ designer. However, much progress has been realized during the past decade to move past the limitations listed above. In general, a maturing of the markets for these methods is occurring. The following list outlines possible reasons for the increase in successfully completed ground improvement projects in highway construction:

1. continued exposure of successfully completed projects to owners and designers;

2. increased familiarity of owners and designers with methods and expected results;

3. identification of an increasing number of qualified specialty contractors;

4. the coordinated effort of contracting agencies, technical societies, universities, and specialty foundation contractors to share and present their research and experience;

5. advancement and acceptance of field testing and instrumentation;

6. the increasing need for contracting agencies to look to these methods as a practical "normal course for consideration" for poor ground conditions rather than a "last resort for consideration."

The following are brief descriptions of various methods of ground improvement and summaries of some of the highway projects in which they have been applied:

Vibro Systems

The primary piece of equipment for construction consists of a vibrating probe or poker approximately 12 to 24 inches in diameter and 10 to 15 feet in length. This probe is capable of generating horizontal vibrations from either electric or hydraulic motors as it is lowered into the ground. Any number of follower tubes can be added to the vibrator to treat soils up to practical depths of approximately 100 feet. A flushing medium of water or air is used to aid in jetting the vibrator into the ground.

The basis for design of vibro systems is the ability of the vibrator to either: 1) densify relatively loose cohesionless soils, known as Vibro Compaction; or 2) reinforce the more cohesive soils, known as Vibro Replacement, by adding stone backfill to the probe penetration holes. This later process creates a "semi rigid stone column" through the soil profile. For many vibro applications for highway work, stone backfill is used to aid in the densification process of the more sandy layers while reinforcing those more silty and /or clayey layers by replacing or displacing them. (See Figure 1)

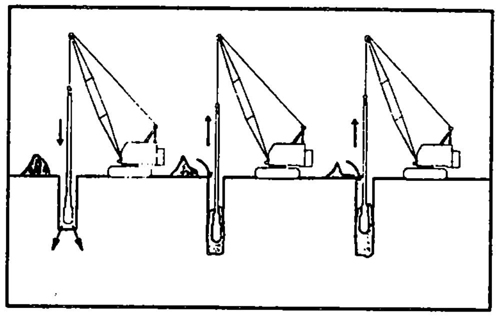

Figure 1 - Vibro Systems Method

The geotechnical considerations addressed by Vibro Systems include: 1) increased bearing capacity; 2) reduced settlement; 3) mitigation of liquefaction potential and lateral spreading; and 4) slope stability.

The original patent and early development of vibrators were made in Germany during the mid 1930's. The introduction and use into North America was made in the 1960's.

Case History No. 1

Alex Fraser Bridge
Annacis Island, British Columbia (1984)
Vibro Replacement

Several Vibro Replacement contracts were performed beneath piers for the northern approach of the bridge. The work was done to mitigate liquefaction in those soils surrounding the bridge piers while the pier loads themselves were founded on driven piles. The stone column points created a densified containment ring or "doughnut" around the piles to provide lateral stability during the earthquake load.

The stone columns were installed to a depth of approximately 65 feet, treating slightly silty to silty fine to medium sands. Electronic cone penetration testing was used to verify that the required improvement (generally Q_c values of 125 tsf) had been achieved to satisfy criteria established by the Ministry of Transportation and Highways.

Case History No. 2

U.S. 42 Over Sugar Creek
Gallatin County, Kentucky (1984)
Vibro Replacement

Construction of a new bridge for highway widening also required soil embankment construction for one of the approaches. The new embankment fill height was approximately 30 feet to match the existing highway elevation. The foundation area for the new embankment, encompassing approximately 15,000 ft^2, was underlain by soft to medium stiff silt and clay. These cohesive soils had been deposited over many years by the contributing flow of water from the Ohio River and Sugar Creek. The geotechnical considerations were driven by slope stability requirements and included improving both the shear strength and drainage capacity of the cohesive foundation soils. (See Figure 2)

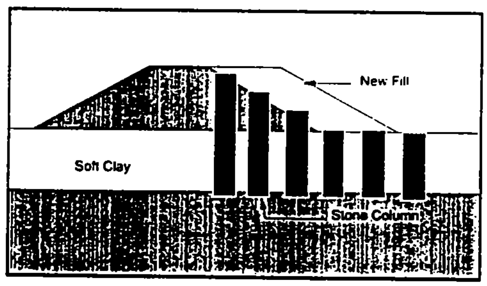

Figure 2 - Embankment Widening with Stone Columns

A total of 7,560 linear feet of stone columns were installed throughout the embankment foundation area to depths ranging form 22 to 37 feet. The spacing and diameter of the stone columns were contractor determined to meet the Kentucky D.O.T. requirements for: 1) improved shear strength of the foundation soils to improve the factor of safety against slope stability failure; and 2) increased drainage paths of the cohesive foundation soils to increase the consolidation rate and reduce the embankment construction schedule.

Case History No.3

I-90 Corridor
Seattle, Washington (1988/90)
Vibro Replacement

During major reconstruction and expansion of Interstate 90 in Seattle, two contracts for stone column installation were completed to improve foundation soils beneath large footings. These footing areas, approximately 30 feet wide by 300 feet long, support a portion of the lid (cut and cover tunnel) structure on the south side of I-90 immediately west of the Mt. Baker Tunnel. The footings were originally to be soil-supported on dense glacial till with a design bearing capacity of 6 KSF. However, during the construction phase, an unanticipated fill, loose to medium dense silty, gravelly sand, was encountered below a portion of the lid foundation. This fill, approximately 20 feet in depth, was determined to be inadequate to support the design loads while limiting settlements to required tolerance. Other alternatives considered and rejected by WSDOT included overexcavation and replacement of the fill, Dynamic Compaction, Vibro Compaction, piles, and drilled shafts.

A total of 9,420 linear feet of stone columns were installed through the fill down to the top of dense fill at approximately 15 to 20 foot depth to both densify and reinforce the silty, gravelly sand fill. The diameter and spacing of the columns were contractor determined to meet a load test criteria established by WSDOT. The work was designed to limit settlement under a plate load test to 0.75 inch under the design 6 KSF load. The plate load test results were then extrapolated to estimate total settlement of the entire soil profile beneath the lid structure.

Extrapolation of the plate load test data was found to reasonably predict actual footing settlement. SPT results showed the stone columns effectively densified the site soils.

Case History No. 4

SR 500 - Andresen Road
Vancouver, Washington (1994)
Vibro Replacement

Vibro Replacement (stone columns) were used to densify loose, clean sands located beneath the bridge pier footings, MSE walls, and adjacent approach embankments for new bridge interchange construction in Vancouver, WA. The geotechnical considerations included, foremost, the settlement of the bridge pier footings (4.5 KSF design load) and secondly, liquefaction mitigation of the foundation soils. More expensive treatments for deep foundations included piles and drilled shafts.

Two areas beneath the bridge footings and embankments, each approximately 125 by 125 feet in plan, were treated by stone column densification to a depth of 20 feet. A total of 12,000 linear feet of stone columns were installed to densify the soil profile, which included both the existing embankment fill soil, as well as the native granular soil. The spacing of the stone columns was contractor designed in accordance with a performance specification. Quality assurance included both SPT testing as well as plate load testing to maintain compliance with WSDOT settlement and liquefaction constraints.

Dynamic Deep Compaction

The Dynamic Deep Compaction method (DDC) involves impacting the ground surface with heavy weights, typically of steel and ranging from 10 to 35 tons. Weights are lifted with standard, modified, or specialty machines and dropped from approximately

50 to 120 foot heights. Energy delivered to the ground is controlled by selecting the weight, drop height, number of drops per point, and the spacings of the drop grid. (See Figure 3)

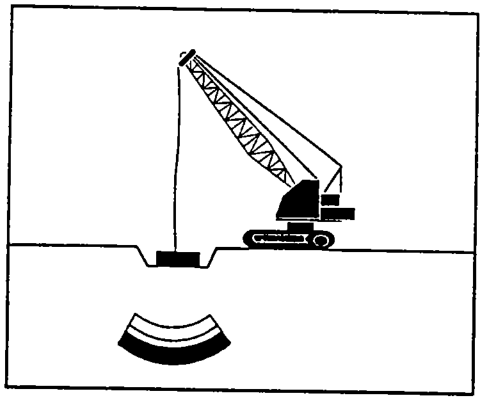

Figure 3 - Dynamic Deep Compaction Method

The principle of dropping heavy weights onto the ground surface to improve soils may be several centuries old as indicated by early drawings and reports. However, the current practice of developing high energy tamping levels with large crawler cranes was first used in France in 1970, Great Britain in 1973 and North America by 1975.

The most widely recognized use of DDC is to densify granular soils. However, DDC also offers a method of compaction or "preconsolidation" for some heterogeneous fills, construction debris fills, and sanitary landfills. The landfill application, first applied to an Arkansas highway in 1982, significantly reduces both total and differential settlements in highway embankment construction. In addition, the method is often attractive in highway work where relocation is impossible, removal is environmentally impractical, and bridging or surcharging is too expensive.

Case History No. 5

US 71
Fayetteville, Arkansas (1982)
DDC

New construction for a four-lane divided highway north of Fayetteville crossed an existing 220,000 yd^3 sanitary landfill. The depth of the landfill varied from 20 to 40 feet. The treatment area of DDC through the right of way was approximately 200,000 ft^2 (4.5 acres). The required energy was developed utilizing a 20-ton weight. Prior to treatment, a coarse granular fill (45,000 tons) was placed over the treatment area to support the crane and to aid in the compaction process. The stone mattress also created a densified raft of granular material that helped bridge over softer zones and reduced differential settlement.

Since conventional testing is not appropriate in a landfill, a large-scale static load test was performed to determine compressibility before and after treatment. Fill soil was placed to an 18 foot height and survey readings made on a settlement plate. The testing indicated that settlement of untreated landfill was 12 inches after 7 days while the treated landfill settled only 1/2 inch after a 7 day period.

Case History No. 6

Lambert/BFI Landfills
Evansville, Indiana (1986)
DDC

In another DDC application similar to the above, construction of a new interchange at US 41 and I-164 near Evansville crossed two landfill areas. One treatment area was underlain by household trash to a depth of 20 to 30 feet. The other area was underlain by 10 to 20 feet of construction debris.

The DDC work again utilized 20 ton weights. Prior to the DDC treatment, a layer of crushed stone was placed over the landfill. Also similar to the Arkansas landfill, the quality assurance included large-scale static load tests. Settlement readings on the untreated landfill were indicted to be approaching 2 feet after 7 days while settlement on the treated landfill was approximately 3 inches after 7 days.

Compaction Grouting

Compaction, or displacement, grouting is a method whereby low slump (less than 2 inches), mortar-type grout is pumped into the ground under pressure (typically from 100 psi to 400 psi, or greater, at the point of injection). The method is most often used to repair compaction deficiencies, both natural or man-made, by first filling any existing voids in the soil profile and then densifying or reinforcing the soil by displacement. In addition, the grout column left in place by the method acts to increase the overall shear strength of the soil mass. (See Figure 4) Another use of compaction grouting is in the arresting of settlement and re-levelling of existing structures. (See Figure 5)

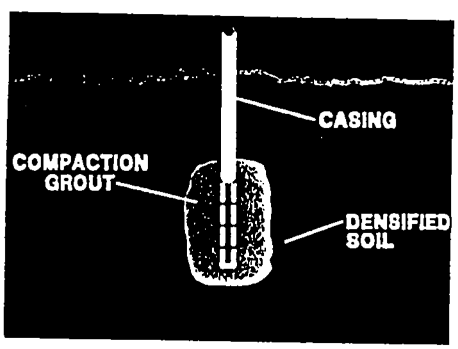

Figure 4 - Compaction Grout (Displacement)

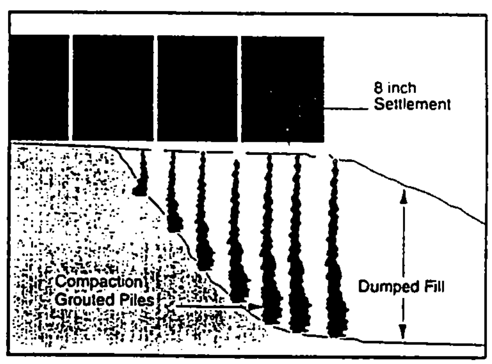

Figure 5 - Compaction Grout Application to Arrest Structure Settlement

The geotechnical considerations most addressed by compaction grouting are settlement reduction, slope stability and liquefaction mitigation. Since compaction grouting is more suited to retrofitting or repairing existing structures, its highway applications include stabilizing and/or improving soils beneath failed pavement sections and the soils surrounding and beneath existing bridge piers and abutments.

The earliest development of compaction grouting dates back to the early 1950's in California, where it was first used as a remedial technique.

Case History No. 7

New River Bridge
Imperial County, California (1989)
Compaction Grouting

Severe seismic induced damage to the Worthington Road Bridge resulted in extended closing of the bridge and a subsequent decision to replace the aging wood pile and frame structure with a 4-span reinforced concrete bridge. However, due to the proximity of the site to a number of active faults, the soils beneath the new replacement bridge foundations required stabilization to increase in situ density and mitigate liquefaction potential. The primary soil layer that required densification was a loose fine sand and silty sand channel deposit encountered beneath embankment fill soils.

Compaction grouting treatment areas extended the width of the approach fills and up to lengths of 300 feet on each abutment. The depth of treatment was 30 feet, at which depth a firm stratum was encountered. A simple quality assurance program included monitoring and recording all grout volumes, pressures, and surface movement to calculate density increases.

Case History No. 8

Green River Bridge (SR 18)
Auburn, Washington (1994)
Compaction Grouting

Highway expansion work for new bridge construction of SR 18 over the Green River also required the "retrofitting" or densifying of potentially liquefiable soils surrounding the existing south bridge abutment. The design intent was to densify a loose granular soil zone that encircled the existing bridge abutment on three sides to reduce the likelihood of lateral soil movement away from the foundation during the design earthquake.

A total of 175 compaction grout points were installed up to depths of approximately 35 feet. Approximately 10,000 ft^3 of compacation grout were injected. Quality assurance again included monitoring and recording of all grout volumes and pressures, as well as SPT test borings performed both before and after the grouting.

Chemical Grouting

Chemical (permeation) grouting involves the injection of low viscosity liquid grout into the pore spaces of granular soils. The base material is typically sodium silicate or microfine cements. In essence, chemical grouting glues sands (less than 15% - 20% silt content) together to form a weak sandstone (shear strengths ranging from 50 psi to 300 psi) for structural support or groundwater control. (See Figure 6) Sleeve-port grout pipes are drilled and grouted in place and internal packer systems are used to inject grout at predetermined packer locations (i.e. 18 to 24 inches). Typical grout pipe spacing ranges from approximately 4 to 8 feet. Highway applications for chemical grouting have typically included: 1) stabilization of soils between the ground surface and new tunnel construction during soft ground tunnelling for settlement control; and 2) arresting on-going settlements of existing bridges and highway structures due to poor soils or scour.

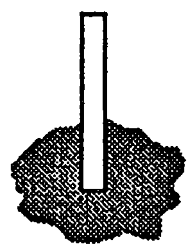

Figure 6 - Chemical Grout (Permeation)

The origins of chemical grouting are from Europe in the early 20th century, with the method generally recognized as a practical geotechnical process by the 1950's.

Case History No. 9

Riverside Avenue Bridge Repair
Santa Cruz, California (1986)
Chemical Grouting

The Riverside Avenue Bridge, spanning the San Lorenzo River in Santa Cruz, was originally built in the 1930's, with abutments and two interior supports founded on piles. During the 1950's, two upstream nose piers were constructed on spread footings founded onto clean sands for the purpose of protecting the bridge piers. The nose piers were tied into the bridge supports by dowels. Scour of the foundation soils beneath one of the nose pier footings had resulted in significant settlement of the nose pier tip, in turn creating damage to the bridge foundation. (See Figure 6)

A 3,200 ft^2 treatment area beneath the settling nose pier was chemically grouted to solidify the sands and reduce the susceptibility to further erosion. Grout pipes were placed through 17 feet of water and sediment and sodium silicate gel/ MC 500 microfine cement was injected. Quality assurance included grout sampling, monitoring and recording of all grout pressures and quantities, and random pressure testing of various sleeve ports to verify thoroughness of permeation.

Case History No. 10

Los Angeles Metro Rail
Contract A-130
Los Angeles, California (1989)
Chemical Grouting

Within a section of the LA Metro Rail System, aproximately 730 feet of twin-tube 21 foot diameter tunnels were designed to pass beneath the 8-lane Hollywood Freeway and a major intersection. The depth of overburden varied from approximately 15 to 20 feet to the tunnel crown with soils consisting primarily of alluvial medium to coarse sands and gravels. An extensive chemical grout program was performed to stabilize the overburden soils and limit ground surface settlement of the freeway and intersection.

A total of 72 horizontal grout pipes (300 foot lengths) and 345 vertical pipes (20 foot lengths) were drilled and installed. A total of 2,300,000 gallons of chemical grout were injected.

Ground surface settlements were observed to be minimal in the grouted sections, well within the design limitations of 1/2 inch. Surface settlements of ungrouted sections of the tunnels were observed to be approximately 3 inches, each trough being approximately 25 feet wide for both tunnels. An unexpected fire in July of 1990 throughout the entire tunnel length burned most lagging support and caused complete collapse of the overburden soils within the ungrouted section of tunnel underlying a vacant lot. The grouted sections remained stable during and after the fire, providing indisputable evidence of the quality and integrity of the chemically grouted zones.

Jet Grouting

Jet grouting refers to the method of employing high pressure erosive jets of water or grout (6000 psi to 8000 psi) to break down the existing soil structure, removing varying portions of the soil and replacing them with cement based grouts. The resulting cylindrical or panel shapes of hardened soils are known as soilcrete, having compressive strengths typically in the range of 500 psi to 1500 psi, depending upon the soil type being treated and the water/cement ratio of the grout being injected. The method offers an alternate to the constraints of permeation grouting, whereby jet grouting can treat a wider range of soil types across the silt range and into some clay soils. Applications for the method include: 1) underpinning; 2) groundwater control;

and 3) tunnels and shafts. Highway applications to date have included: 1) underpinning and stabilization of settling bridge piers due to scour; and 2) groundwater control and excavation support for utility trench crossings beneath roadways; and 3) stabilization for soft ground tunnelling in cohesive soils beneath roadways.

The origins and development of the method are from Japan during the early 1970's, extending to Europe in the late 1970's. Early jet grouting work in the U.S. was performed in the mid-1980's.

Case History No. 11

Auhai Street/Utility Protection
Honolulu, Hawaii (1990)
Jet Grouting

The City of Honolulu's Department of Wastewater Management required placement of a new sewer utility with the invert at a depth of 19 feet below existing grade within a busy downtown intersection. The new utility consisted of 72 and 48-inch reinforced concrete pipes and associated junction structures. The soil profile below surface asphalt and base material, consisted of very loose silty sands to a depth of 17 ft, where hard coral was encountered (the water table existed at a depth of approximately 2 feet). The excavation depth for the new sewer was to a depth of 17 feet.

Conventional open-cut excavation with sheet piles was employed on the project by the general contractor. The sheet piling operation was disrupted at the intersection of Auhai Street and Ward Avenue, where numerous existing utilities crossed the new sewer alignment. Relocation of the existing utilities was not possible. The jet grouting technology was used to create a cofferdam for the proposed excavation cut. This cofferdam provided groundwater control and excavation support around and beneath all existing utilities, while eliminating the need to relocate them. In addition to consideration of all utilities, the soilcrete column layout was designed to tie into the sheet piling already in place.

Case History No. 12

Salt River Canyon Bridge - U.S. 60
Arizona (1993)
Jet Grouting

During construction of the Salt River Canyon Bridge, heavy rains pushed the river's water more than 30 feet above normal levels, high enough to surround the new bridge abutment. The site is located on US Route 60, 120 miles east of Phoenix, AZ. Most of the canyon walls underneath the bridge are sheer vertical rock faces. An area immediately adjacent to the new abutment, thought to be a solid portion of the canyon wall, scoured out, leaving a 20-foot wide gouge in the rock. The gouge threatened the stability of the new bridge and washed away the only access road to the recreational areas along the river's edge. The gap became visible when the river waters receded.

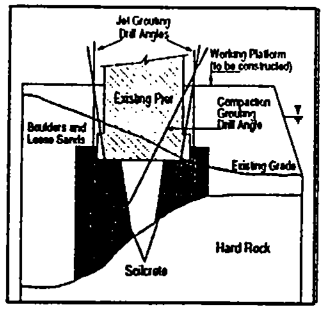

Figure 7 - Jet Grouting Application for Bridge Foundation Scour

The task required placing retaining walls, thereby protecting the new bridge abutment. A mass of soilcrete was jet grouted below the normal river level, filling the gap and acting as an arched retaining wall, stabilizing the scour zone. (See Figure 7). Following this work, a concrete retaining wall was founded on top of the soilcrete mass to fill the gap above water level. Restoration of the access road was then completed. During jet grouting, care was taken to keep the work site clean and all spoil material contained, since the job site was within a scenic recreational area.

Lime/Fly Ash Slurry Injection

The method involves injecting a lime/fly ash mixture into foundation subgrades and slopes, typically at pressures of 50 psi to 100 psi and depths up to 15 feet for highway work and 40 feet for railroads. Lime/fly ash slurry injected into a subgrade or slope will displace trapped water and impede re-entry. (See Figure 8) As cracks, weakness planes, and voids are filled, the subgrade structure is strengthened. The method has proven effective in fine grained soils, including expansive clays and water sensitive silts. Highway applications include: 1) stabilization of pumping and/or settled pavement sections; and 2) relatively shallow slope stability failures. The method had its origins and development from the late 1970's in Texas, where it was first used to reduce volumetric changes in expansive clays. (See Figure 8)

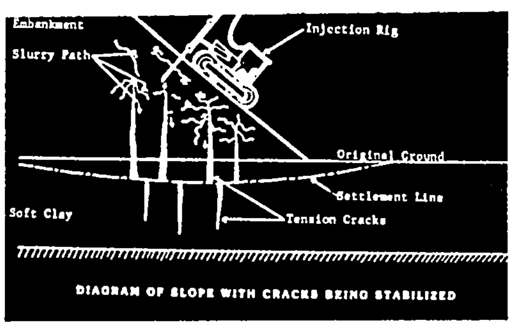

Figure 8 - Lime/Fly Ash Slurry Injection Method

Case History No. 13

Beloat Road
Maricopa County, Arizona (1990)
Lime/Fly Ash Slurry Injection

County officials discovered during a road improvement program in western Maricopa County that a one-mile section of Beloat Road was underlain by a water-sensitive silt. The one-mile section of roadway was bordered on each side by irrigated farmland, which contributed to the problem. During rainy periods the road was virtually impassable. Other areas of the road were experiencing pumping due to underground water intrusion.

Although other remedial methods were tried, including adding river rock and geotextiles, neither method was successful in allowing for completion of the subgrade compaction. Lime/fly ash injection was selected as the most cost-effective method to quickly solve the problem. A length of 5,000 feet of the county road was injected to a depth of four feet. The lime/fly ash was double injected from curb to curb (two passes), 32 feet wide. By injecting lime/fly ash, a moisture barrier was created in the subgrade of the soil. The network of the lime/fly ash seams served to increase the shear strength of the soil as well as to reduce the moisture content by forcing out free water. After the injection was completed, the remaining slurry on the surface was scarified into the ground to create a firmer working subgrade. The injection was successfully completed in three weeks, allowing Maricopa County to finish the delayed paving program.

Case History No. 14

Loop 499
Harlingen, Texas (1993)
Lime/Fly Ash Slurry Injection

In Harlingen, the Texas Department of Transportation was widening Loop 499, a two-lane highway built over a landfill. Constructed in the 1970's, the highway had continually settled and repeatedly been resurfaced. The years of maintenance created a four foot buildup of asphalt in some areas. Lime/fly ash injection was used to stabilize the soil beneath the existing and future traffic lanes. The project required working in and around traffic on the existing highway.

To reach the required 28 foot depth, a specialized injection unit was used with rotary-percussion drills to pre-drill the asphalt prior to injection. Lime/fly ash injection forced out groundwater and filled voids, while increasing the landfill's density, giving it greater strength and helping to impede future settlement. To verify a reduction in the rate of settlement of the landfill, ongoing monitoring of the pavement surface elevations is being performed.

CONCLUSIONS

The ground improvement case histories presented in this paper represent a partial list of completed projects and are taken from a much larger body of work in highway construction. This volume of work has increased significantly over the past decade. In addition, the methods have been adapted to increasingly difficult geotechnical and site requirements. Based upon these points, ground improvement will continue to be an important contributor to highway construction in the coming decade.

References

1. Allen, T.M., Harrison, T.L., Strada, J.R., and Kilian, A.P., "Use of Stone Columns to Support I-90 Cut and Cover Tunnel," Deep Foundation Improvements: Design, Construction, and Testing, ASTM STP 1089, American Society for Testing and Materials, 1991.

2. Gularte, F.B., Taylor, G.E., Monsees, J.E., and Whyte, J.P., "Tunnelling Performance of Chemically Grouted Alluvium and Fill," Los Angeles Metro Rail, Contract A-130, RETC Conference, Seattle, WA 1991.

3. Hayden, R.F., "Utilization of Liquefaction Countermeasures in North America," 5th National Earthquake Conference (EERI) - Chicago, IL, 1994

4. Hayward Baker Project Summaries & Ground Modifacts, various.

5. Klinedinst, G.L. and DiMaggio, J.A., "Pitfalls in Implementing New Technology," Geotechnical Seminar, Central Pennsylvania Section - ASCE, 1983.

6. Koelling, M.A. and Ringen, A.R., "Jet Grouting - Soil Improvement Case Histories," 28th Symposium - Engineering Geology and Geotechnical Engineering, Boise, ID, 1992

7. Moseley, M.K. et al, Ground Improvement, CRC Press Inc., Boca Raton, FL (1993)

GEOSYNTHETIC REINFORCING OF HIGHWAY EMBANKMENTS

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Abstract

Geosynthetic reinforcing of highway embankments is proving to be an economical alternative to constructing costly reinforced concrete retaining walls. This paper gives two California case histories of applications of geotextiles and geogrids in reinforcing earth slopes and describes the design and construction. At Willow Pass near Concord, California, earth slopes as steep as 0.8 horizontal to 1 vertical and with heights up to 36 feet were reinforced with geosynthetics and without structural facing elements. Staged construction required a 36 foot high temporary slope with an inclination of 0.8:1. The slope is subject to truck traffic at the top and is designed for 0.15g seismic force. Geotextile filter fabric was installed in layers in the embankment as it was placed. The forms used to wrap the face with fabric are described and photographs of the construction are included. The exposed fabric is visibly in good condition after two years of exposure. In another case, in Richmond, California, a 30-foot high approach fill to a highway bridge had to be steepened to an inclination of 1:1. This was too steep for the soil conditions and the design seismic loading. A concrete retaining wall was considered but it would have to be supported on long piles because of soft soil conditions. The better solution was to reinforce the slope with geogrids. Global stability pointed to treatment or removal of the soft soil layer but excavation or treatment would mean costly disposal of contaminated soils and groundwater. The soft layer was strengthened by driving short piles across the potential shear plane. Design of geosynthetic reinforced slopes is an extension of conventional slope stability analyses. Specifying geosynthetics should be given careful attention in the selection of factors of safety in order to obtain bids based on equivalent performance.

Introduction

Geosynthetics are playing an increasing role in civil, geotechnical, and geoenvironmental engineering. Twenty five years ago they were virtually unheard of, with some exceptions such as pond liners. Over this period, geosynthetics have gradually gained acceptance and now they are so widely used that probably every new project small and large has incorporated geosynthetics to some extent. There are so many geosynthetic products and systems available that the design engineer is challenged to keep up with all of the technological advances. Preparing proper specifications is the main challenge, especially for highway projects where trade names cannot be specified. This paper includes a discussion of specifying the strength of the geosynthetic reinforcing.

Highway projects are increasingly confined by right-of-way restrictions resulting in the need for steeper slopes or retaining walls. Geosynthetic reinforced slopes and retaining systems are less costly alternatives to conventional concrete cantilever retaining walls because:

- concrete walls need to have large footings, formwork, costly steel reinforcing and concrete, and

- geosynthetic reinforced slopes need only thin geosynthetic reinforcing elements embedded in the backfill as it is placed.

This paper next describes the design, the geosynthetic properties, the specifications, and the construction of two projects requiring steep slopes.

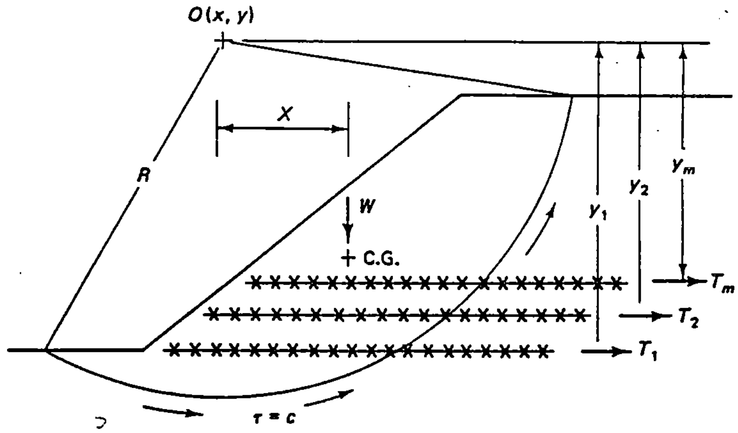
Design of geosynthetic reinforcing

The analysis of a reinforced slope begins with a limit equilibrium slope stability analysis (Figure 1). When the factor of safety is low for the internal, surcharge, and seismic loads, and for the soil strength; tensile reinforcing forces can be introduced in the analysis to improve the factor of safety. Many slope stability computer programs provide for tensile or tie-back forces in the analysis.

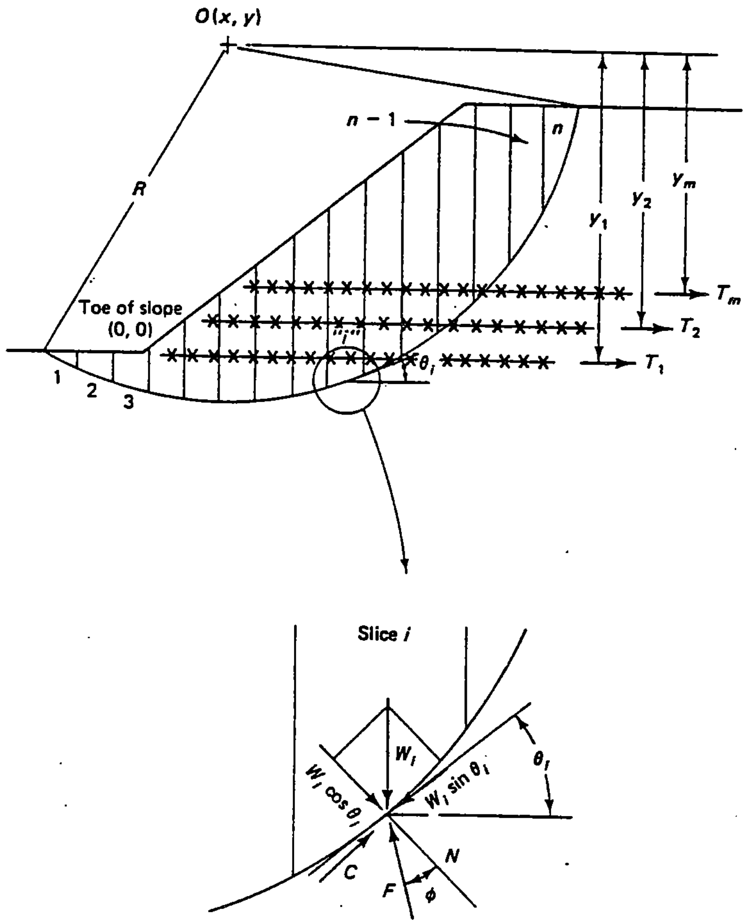
From the analysis, the contribution of the reinforcement to the factor of safety can be determined. The allowable tensile strength requirement of the geosynthetic reinforcing can then be determined. Multiple layers of reinforcing can be introduced into the slope stability analyses. The vertical spacing should be wide as practical to minimize the labor of placement and handling. For slopes, 30 to 42 inch vertical spacing is common. Vertical spacing requirements for geosynthetic reinforcing of vertical walls will be less.

Geotextile anchorage length behind the failure zone can be determined using the relationship shown on Figure 2 .

External global stability of the geosynthetic reinforced mass can be verified by checking for the factor of safety against sliding, overturning, and bearing capacity. These factors should be 1.5, 2, and 3, respectively, for permanent conditions. If the improvements at the

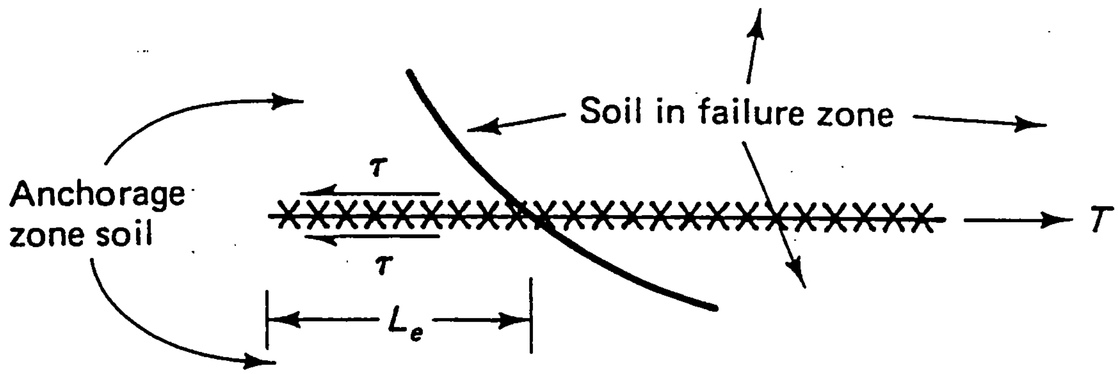


Details of circular arc slope stability analysis for soil strength represented by undrained conditions.



Details of circular arc slope stability analysis for (c, φ) shear strength soils.

Figure 1: Slope stability analysis of a reinforced slope (after Koerner, 1990)



$$\Sigma F_x = 0; \quad 2\tau E L_e = T(FS)$$

$$2(200)\,(0.8)L_e = 250(12)\,(1.5)$$

$$L_e = 14.1 \text{ ft.}$$

Figure 2: Determination of geotextile anchorage length behind the failure zone (after Koerner, 1990)

top of the slope are settlement sensitive, the settlement of the sloped embankment should also be estimated.

Specifying The Geosynthetics

The allowable tensile strength is computed from the above analysis. In the case of steel reinforcement for structures, the yield or ultimate strength is specified, not its allowable strength. For geosynthetics we should not simply apply a factor of safety and specify the ultimate strength of the geosynthetic because there are many geosynthetic materials with different physical and chemical properties that can provide the required tensile strength. These various materials should have different factors of safety applied because they have different properties under different environmental conditions. The objective of a good design is to open up the field to obtain the lowest bid on geosynthetic reinforcing, be it geogrid, geofabric, metal strips, welded wire or any product that will provide the equal specified performance, including longevity. Different materials will meet the minimum criteria but the factor of safety may be different for various geosynthetics. So, the required factor of safety or the method of obtaining the factor of safety is specified for a given application. This results in different ultimate tensile strength requirements for different geosynthetic products.

In summary, the (1) T_{allow}, (2) the test for ultimate tensile strength, usually ASTM 4595, and (3) the factors of safety against installation damage, creep, chemical degradation, and biological degradation are specified. T_{allow} is specified by the equation shown on Table 1 and each term is defined and specified in the contract documents.

Case History - Route 4, Willow Pass Grade Lowering Project

The project consisted of flattening the steep vertical alignment of the existing Route 4 across the hills between Concord and Pittsburg, California. It had to be flattened because the BART train tracks would be placed in the median of Route 4. The trains cannot navigate the steeper highway traffic grades. In lowering Route 4 through the hills, the excavated hillside material was used to raise the grade in the lower portion of the project. To maintain traffic, the fill had to be placed on one side at a time, while construction was done on the other side. Due to the high embankment and the limited right-of-way, there was not sufficient space, between the temporary trafficked lanes and the new construction above, for a safe slope incination of 2:1 or even 1.5:1. There was space only for a slope inclination varying from 1.5:1 to 0.8:1.

Several alternatives were considered. A reinforced concrete retaining wall with an upsloping backfill would be costly, $35 to $40 per square foot (psf), for a temporary situation. Similarly, a reinforced earth wall with metal strip reinforcing and concrete

Allowable Versus Ultimate Geotextile Properties

RECOMMENDED PARTIAL FACTOR OF SAFETY VALUES FOR USE IN EQUATION 2.18

Application area	Various partial factors of safety: Installation damage	Creep*	Chemical degradation	Biological degradation
separation	1.1 to 2.5	1.0 to 1.2	1.0 to 1.5	1.0 to 1.2
cushioning	1.1 to 2.0	1.2 to 1.5	1.0 to 2.0	1.0 to 1.2
unpaved roads	1.1 to 2.0	1.5 to 2.5	1.0 to 1.5	1.0 to 1.2
walls	1.1 to 2.0	2.0 to 4.0	1.0 to 1.5	1.0 to 1.3
embankments	1.1 to 2.0	2.0 to 3.0	1.0 to 1.5	1.0 to 1.3
bearing capacity	1.1 to 2.0	2.0 to 4.0	1.0 to 1.5	1.0 to 1.3
slope stabilization	1.1 to 1.5	1.5 to 2.0	1.0 to 1.5	1.0 to 1.3
pavement overlays	1.1 to 1.5	1.0 to 1.2	1.0 to 1.5	1.0 to 1.1
railroads	1.5 to 3.0	1.0 to 1.5	1.5 to 2.0	1.0 to 1.2
flexible forms	1.1 to 1.5	1.5 to 3.0	1.0 to 1.5	1.0 to 1.1
silt fences	1.1 to 1.5	1.5 to 2.5	1.0 to 1.5	1.0 to 1.1

*The low end of the range refers to results that have been compensated for creep in the performance of the tests.

$$T_{\text{allow}} = T_{\text{ult}} \left(\frac{1}{FS_{ID} \times FS_{CR} \times FS_{CD} \times FS_{BD}} \right) \tag{2.18}$$

where

T_{allow} = the allowable tensile strength,
T_{ult} = the ultimate tensile strength,
FS_{ID} = the factor of safety for installation damage,
FS_{CR} = the factor of safety for creep,
FS_{CD} = the factor of safety for chemical degradation, and
FS_{BD} = the factor of safety for biological degradation.

Table 1: Determination of allowable tensile strength of geosynthetics (after Koerner, 1990)

panels is a permanent wall and also would be costly, $25 to $30 psf, for temporary retainment of fill, later to be buried. A geogrid reinforced slope with buried plastic grids would be a better solution but to bench and tie the second phase fill into the first phase fill slope would be more difficult with the stiffer plastic reinforcing than the fabric layers. The fabric reinforcement can be torn by the grading equipment in the benching operation without disturbing the deeper reinforcement.

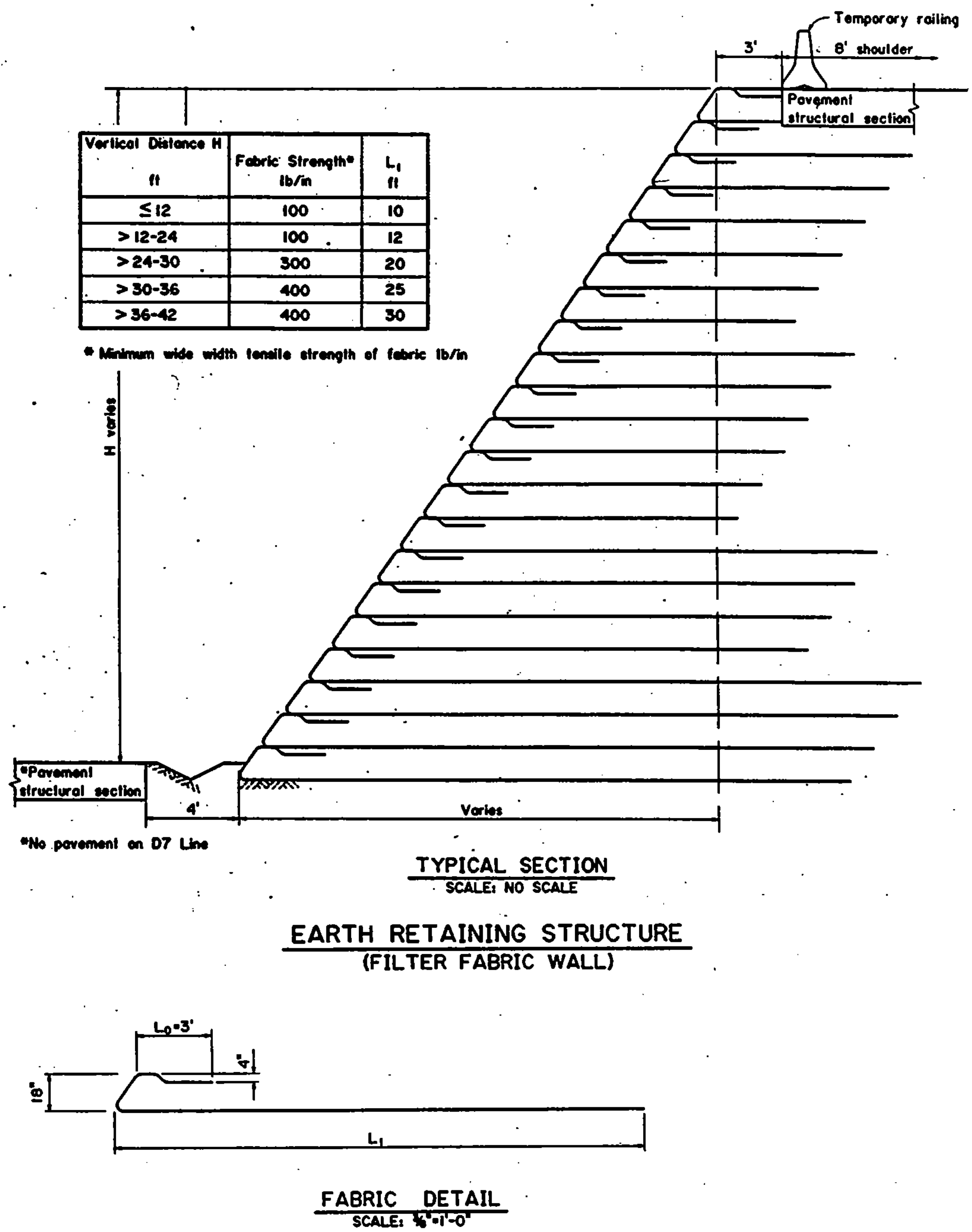
The fabric reinforced slope was the lowest cost solution at about $8 psf of the vertical projection of the slope. However a cost comparison should be on a lineal foot basis because the retaining wall heights would be about half the slope height with an upsloping embankment behind the wall. For a 36 foot high embankment, the comparative costs would be approximately $700, $500, and $300 per lineal foot, respectively for the concrete cantilever wall, the reinforced earth wall, and the fabric reinforced slope.

The fabric reinforced slope as designed is shown on Figure 3. The new roadway is 36 feet higher than the old roadway and the limited horizontal distance between results in slopes up to 0.8 to 1. These are too steep for an earth slope to stand when subjected to heavy truck loading, a 0.15g design seismic load, and 2 years of winter rains. The siltstone from the hillside cuts broke down into a silt during compaction. The design called for 100 to 300 pound/lineal inch (pli) tensile strength at vertical spacing of 18 inches. The length of the reinforcing varied from 25 ft at the bottom to 12 feet at the top. Lengths were determined in design by analyzing potential failure surfaces with PCSTABLM at various levels up the slope. Reinforcing was provided as necessary to provide a minimum factor of safety of 1.25 against slope failure at any level.

A general view of the reinforced embankment is shown on Figure 4. The contractor proposed a modification to the design shown on the drawings. The modification is shown on Figure 5 and consisted of increasing the vertical spacing from 18 inches to 30 inches and using higher strength fabric. He proposed woven propylene of 600 pli tensile strength instead of the design 300 pli with similar design lengths. The modification was checked and approved. Exxon GTF 1225 woven polypropylene was used.

Where slopes were inclined steeper than 1:1 they were wrapped with the fabric. The wrapping is to prevent sloughing of soils on steeper slopes. The wrapping is not for stability. The wrapping of the slope face with fabric was accomplished with the forming system shown on Figures 6 and 7. Where the slopes were inclined between 1:1 and 1.5:1, they were not wrapped; the fabric was terminated at the surface of the slope as shown on Figure 8.

After the traffic was shifted to the new east-bound pavement at the top of the first phase fill, the second phase fill will be placed against the fabric-faced temporary slope. The exposed fabric is visibly in good condition after two years exposure. The fill will be benched into the fabric face and compacted as the fill is placed, leaving the horizontal



Vertical Distance H ft	Fabric Strength* lb/in	L_1 ft
≤12	100	10
>12-24	100	12
>24-30	300	20
>30-36	400	25
>36-42	400	30

* Minimum wide width tensile strength of fabric lb/in

Figure 3: Fabric reinforced slope design, Willow Pass Grade Lowering Project, California

Figure 4: Fabric reinforced embankment, Willow Pass Grade Lowering Project, California

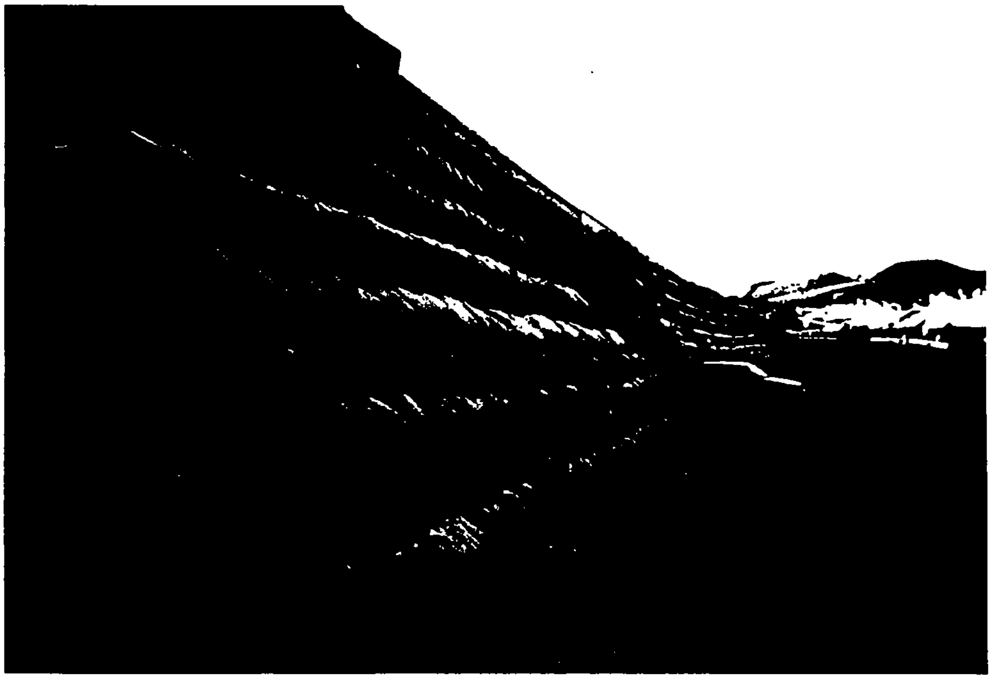


Figure 5: Wrapped face of fabric reinforced embankment, Willow Pass Grade Lowering Project, California

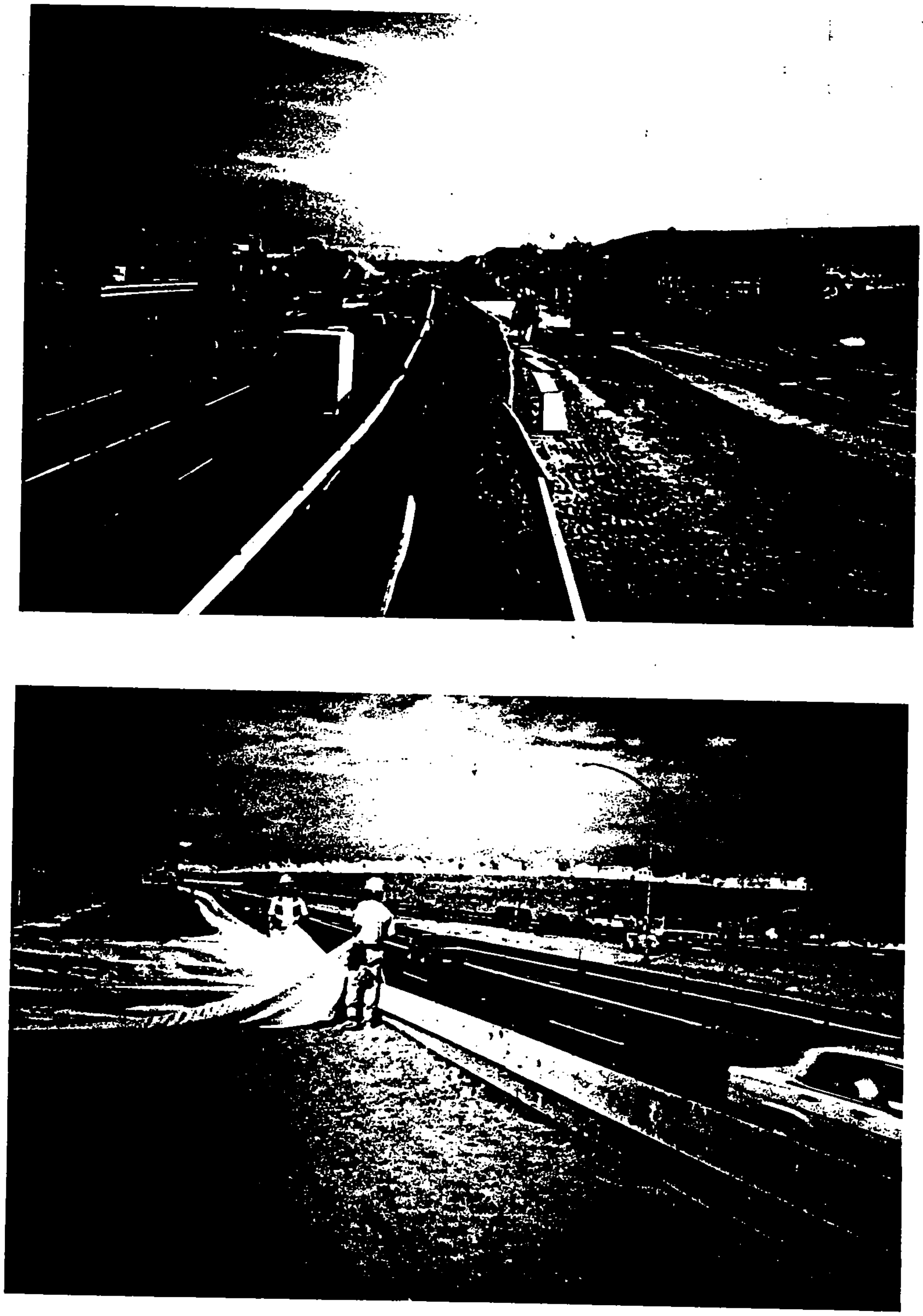


Figure 6: Wood forms for wrapping the face of the fabric reinforced embankment, Willow Pass Grade Lowering Project, California

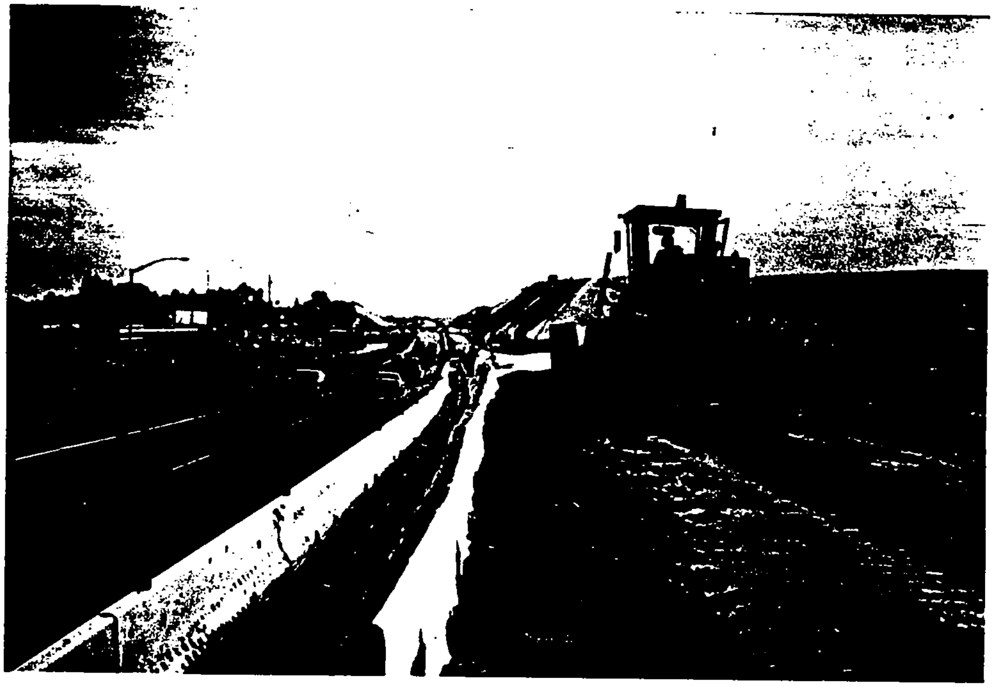


Figure 7: Placing fill against wrapped face of the fabric reinforced embankment, Willow Pass Grade Lowering Project, California

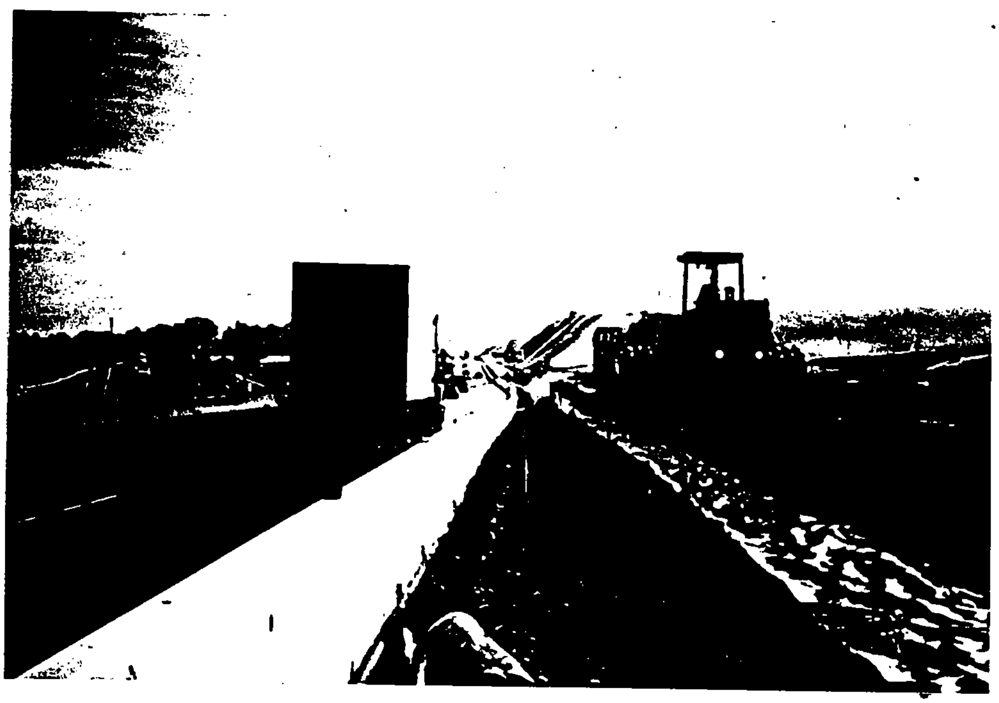


Figure 8: Placing fill on fabric reinforcement of non-wrapped face of embankment, Willow Pass Grade Lowering Project, California

reinforcing buried in place. After the fill is completed, the BART tracks will be placed in the median over the buried temporary slope, and the west bound pavements will be constructed.

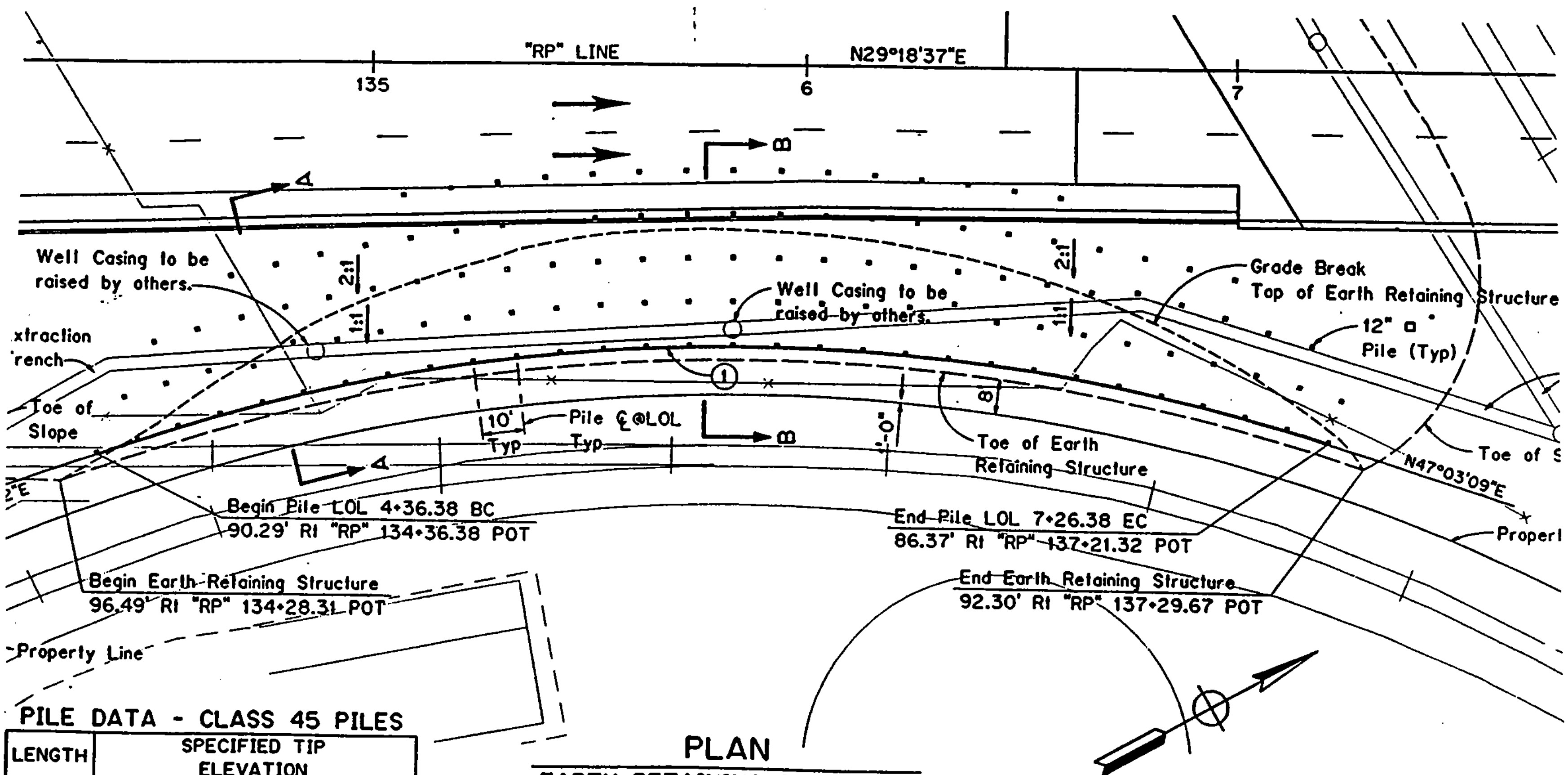
Case History - Richmond Parkway

The Richmond Parkway in Richmond, California, is being constructed over low-lying marshland adjacent to the Chevron Refinery. As shown on Figures 9 and 10, a short section of a 30-foot high approach fill to a railroad overcrossing bridge had to be steepened to an inclination of 1:1 because of the proximity of railroad tracks near the base of the fill. A concrete retaining wall was considered for retaining the lower part of the slope but the wall would have to be pile supported with long piles because a layer of soft Bay Mud underlies the site. A more economical solution was to reinforce the slope, which had to withstand seismic loading, with geogrids placed in the embankment. Global stability required strengthening or removal and replacement of the soft Bay Mud, but excavation or treatment would mean costly disposal of contaminated soils and groundwater. The soft layer was strengthened by driving short piles across the potential shear plane, a brute force but less costly solution than handling and disposing of the contaminated materials.

The specified reinforcing consisted of Tensar UX 1500 primary geogrid with a tensile strength of 2000 plf at a vertical spacing of 36 inches. The length was a constant 20 feet. In lieu of wrapping the face as designed, the contractor obtained approval to install two intermediate lighter 4 foot long Tensar BX 1100 secondary geogrids at a vertical spacing of 12 inches between each primary UX 1500 grid. The secondary intermediate grids were installed only to prevent sloughing; the primary geogrid provided the design tensile strength. Upon close examination, the ends of the geogrids are visible on the face of the slope. The cost of the permanent geogrid reinforcing, including the installation and on-site backfill, for 1:1 slopes runs approximately $10 to $12 per square foot of vertical projection.

Conclusions

Geosynthetic reinforcing of highway embankments is an economical alternative for constructing steep slopes versus other solutions. Savings using geotextile fabric reinforced temporary slopes versus conventional walls at Willow Pass were up to $400 per lineal foot of steepened slope. At Richmond Parkway, similar savings were made by using geogrid reinforcing in a permanent 1:1 slope. The analysis of geosynthetic reinforced slopes is an extension of conventional slope stability analyses. Careful attention should be given to the specifying of the geosynthetics and their factors of safety in order to obtain bids based on equal performance.



PILE DATA - CLASS 45 PILES

LENGTH	SPECIFIED TIP ELEVATION
20'	-14.5'
25'	-19.5'

Figure 9: Plan view of geogrid reinforced embankment, Richmond Parkway project, Richmond, California

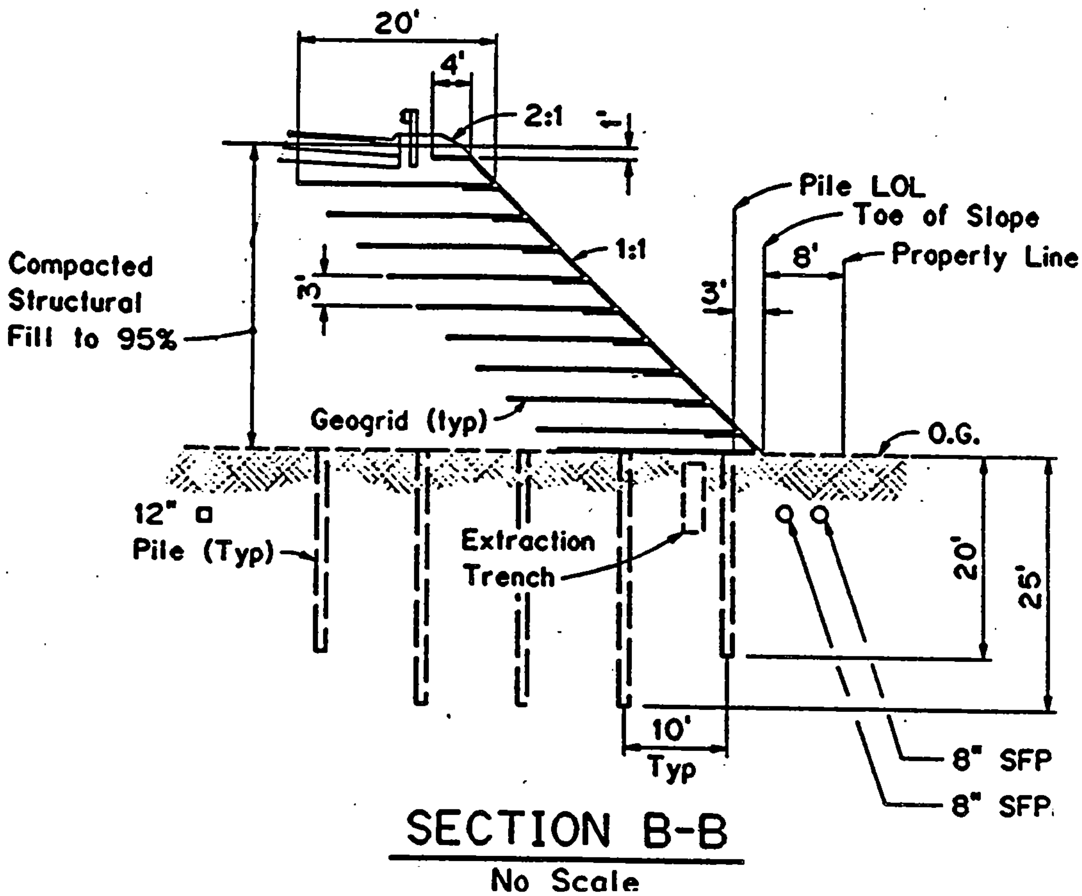


Figure 10: Section of geogrid reinforced embankment, Richmond Parkway project, Richmond, California

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References

Koerner, Robert M., 1990, "Designing With Geosynthetics," 2nd Edition, Prentice Hall

Federal Highway Administration, January 1993, "Guidelines for Design, Specification, & Contracting of Geosynthetic Mechanically Stabilized Earth Slopes on Firm Foundations," Publication No. FHWA-SA-93-025

Application of MSE (Mechanically Stabilized Earth) Slopes as Replacement for Retaining Walls; A New Hampshire Case History

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Abstract

Route 3A, in Hooksett, New Hampshire, was widened in 1990 to improve traffic flow and increase safety. One portion of the project required an earth retention system to compensate for grade changes along the new road alignment. Originally a mechanically stabilized earth (MSE) concrete-faced retaining wall was considered to meet these requirements. The D.O.T., being concerned about costs and aesthetics, explored other types of MSE structures.

New Hampshire D.O.T. chose to use a geogrid-reinforced slope having a maximum height of 43 feet and an average height of 34 feet. The angle of the slope face is 39 degrees (1.25H:1V) at its steepest point. The project was competitively bid by pre-approved geogrid manufacturers and suppliers with in-house engineering capabilities. Sealed engineering drawings and calculations were provided to the D.O.T. prior to the issuing of bid documents. The construction drawings were then included in the bid packages provided to the contractors.

This paper will discuss the application of MSE slopes as a replacement for retaining walls in road widening applications and as an alternative system in general highway construction.

Introduction

Mechanically stabilized earth technology has been used as a construction standard for highway work in a large portion of the U. S. and Canada for the last quarter century. The traditional materials involved have been steel strap or grid structures laid horizontally in granular, free draining soil to create a large, reinforced soil mass. This mass then acts as a large gravity retaining structure designed to have sufficient factors of safety against sliding, overturning, and bearing capacity failures. Also taken into account for these steel-based systems is their propensity for in-situ corrosion. A factor of safety (which results in an increase in the cross-sectional area of the reinforcing elements) for this decomposition must be included in the design of the system. The backfill also has to follow strict gradation and electro-chemical specifications to further mitigate corrosion. The steel reinforcing elements are then attached to a concrete facing panel. Although they are significantly less costly than cast-in-place concrete retaining walls, they have a similar appearance and many of the same limitations.

With the advent of high density polyethylene (HDPE) and polypropylene (PP) geogrid reinforcing elements, MSE technology has significantly changed. Geogrids have allowed this technology to evolve from the confinement of vertical, flat-faced, concrete walls requiring structural backfill, to the use of on-site materials for construction of earth retaining systems with sloping, vegetated facings. These polymeric geogrid materials have high strength in their manufactured form and are not susceptible to in-situ corrosion or decomposition, making them very reliable, structural elements for the 50-100 year design lives these structures require. This evolution has opened up the private sector to the aesthetic and economic advantages of using MSE steepened slopes. Based on some 10 years of success in private development, heavy industry, rail, and a number of experimental, monitored projects sponsored by the FHWA and highway departments around the country, some states have begun adopting this technology as the alternate of choice for their grade separation needs. New Hampshire is among the first to utilize MSE slope technology.

The original concept for the structure required for the Route 3A project described herein was the more traditional, steel-based type of MSE system. When the steepened slope option was presented to the D.O.T., they made a thorough evaluation of the materials, design methodology, and costs associated with the geogrid system and decided to go forward with a reinforced, steepened, slope system.

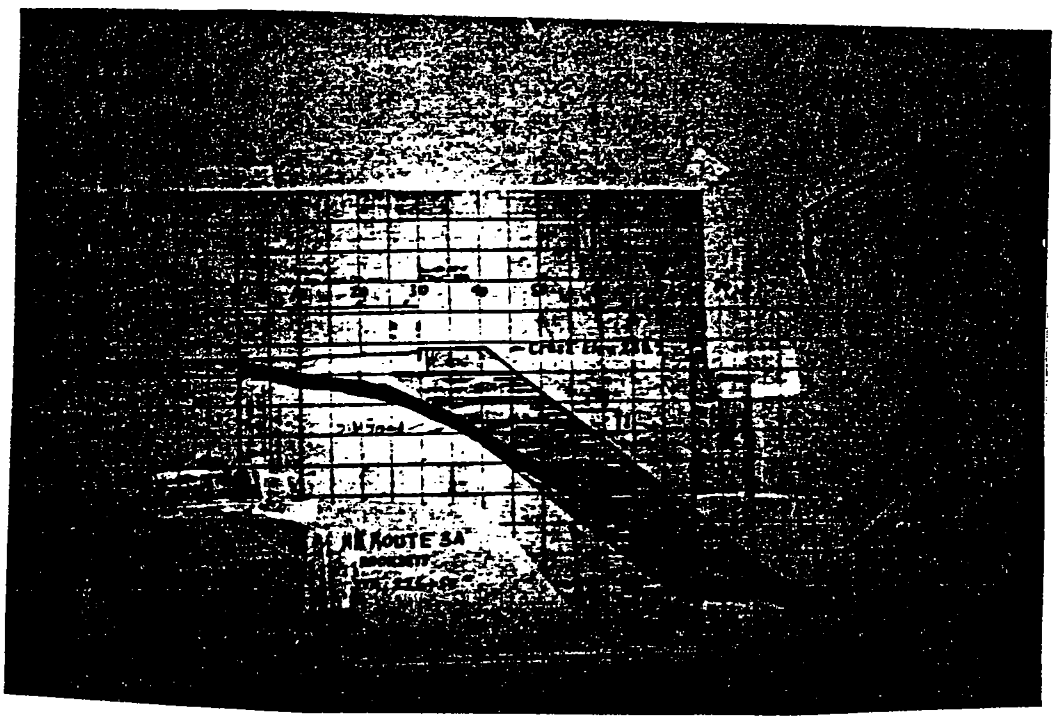
Project Description

Route 3A runs through the town of Hooksett, NH and is just east of Interstate 93. The area is growing due to its proximity to Manchester Airport and several of New Hampshire's larger cities. The area is also about an hour's drive from Boston. The popularity of this region contributed to making the intersection of Route 3A and Main St. heavily used, creating traffic congestion and safety problems. The solution to these problems was to widen the intersection and add a traffic light. This called for a grade separation 225' long and up to 43' high on the east side of the intersection. Right-of-way restrictions forced the D.O.T. to use a retaining structure instead of a 2:1 embankment. Further complicating the grade separation problem was the proximity of an abutting home to the highway right of way. This home is within 10' of the toe of the embankment, making the abutter's quality of life crucial in selecting the type of retaining structure used for the project.

The work was bid using engineering drawings and construction specifications provided by two pre-approved proprietary geogrid system suppliers. These drawings and specifications were included with the bid documents and provided details as to the location, length, and type of reinforcing elements required to complete the work. Construction of the project took about three weeks during July and August of 1990.

Design of the structure

The design provided for the structure by Tensar Earth Technologies, Inc. utilized three different types of products to stabilize the embankment. Uniaxially oriented geogrids (high strength grids with strength in one principal direction) were placed horizontally within the embankment with the principal axis perpendicular to the slope face. These were designed with a height-to-length ration of about .6H and required 100 % coverage of the lift every 4 feet vertically and provide the soil mass with the structural integrity required to stabilize it against the forces which cause overturning, sliding, bearing capacity, and global stability failures. In between these main reinforcing grids, 4 foot wide biaxially oriented (strength in both directions), comparatively low strength grids were placed on 1 foot centers vertically to provide intermediate reinforcement of the slope face. This intermediate reinforcement facilitates construction and protects the structure against sloughing both during and after the work is completed. A crown vetch/tall fescue grass mixture was hydroseeded onto the completed structure. A coconut-based erosion control matting was used to provide the structure with erosion protection until the establishment of vegetation.

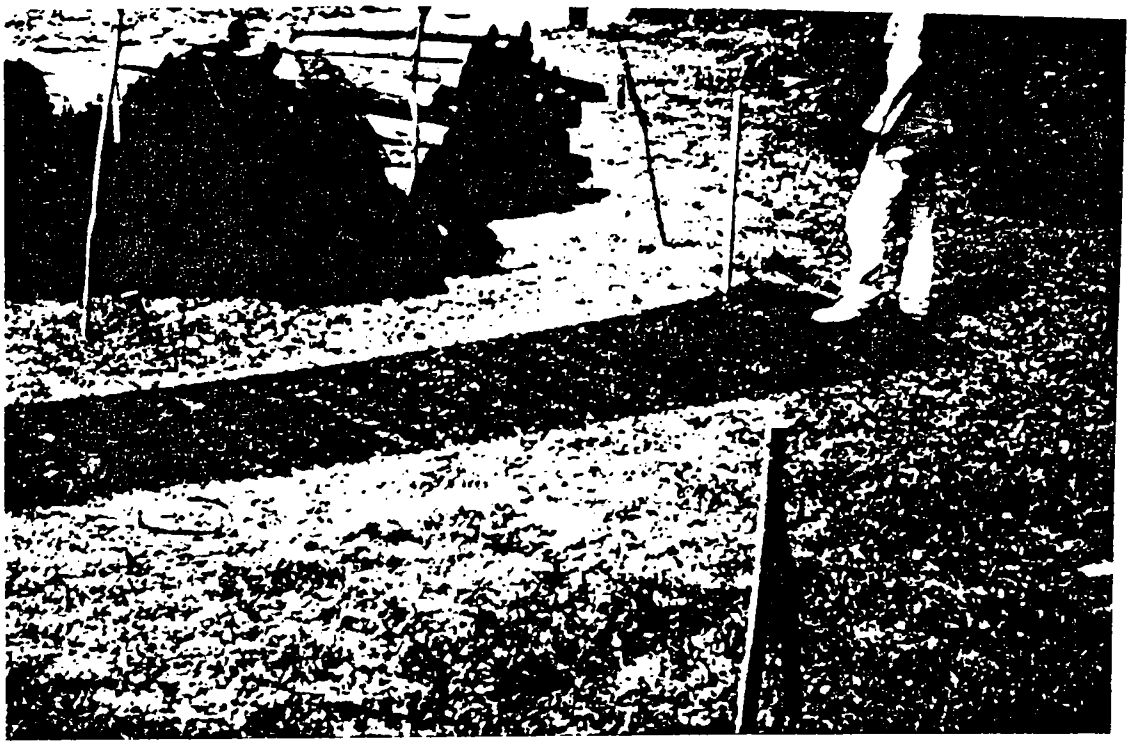


Typical Embankment cross-section.

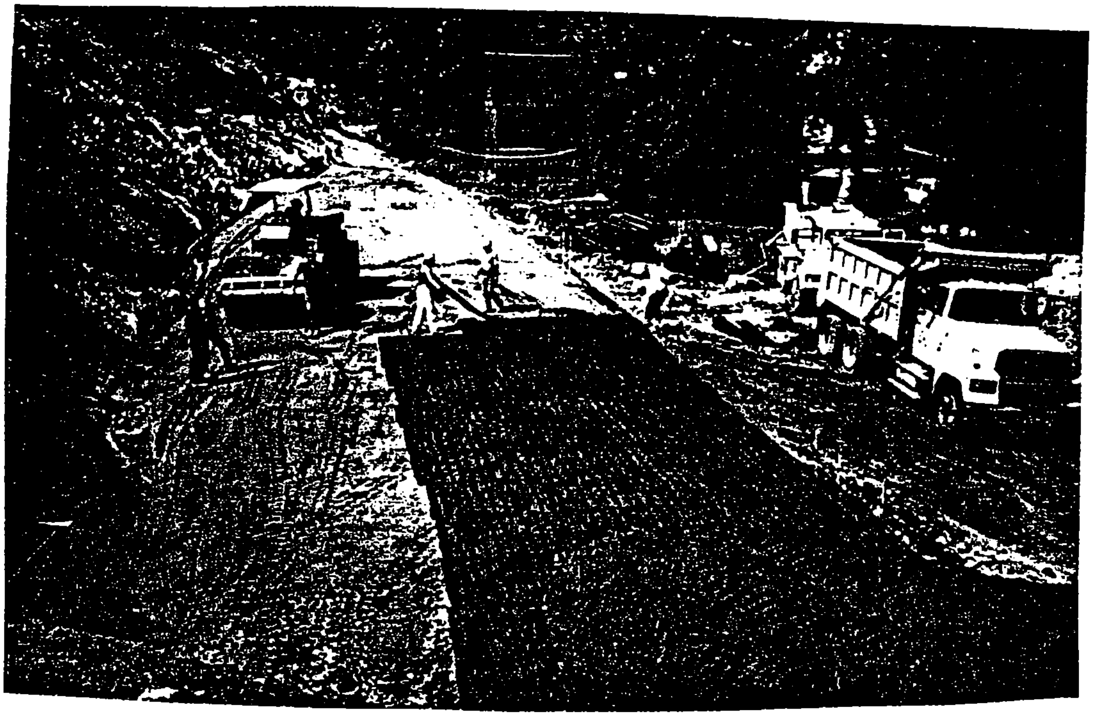
Construction of the MSE slope

Due to the constraints of the site, the contractor had a one-way access problem which complicated construction of the embankment. The excavation requirements for the MSE system were relatively shallow compared to the over-excavation required to create a working platform for the equipment. After completing this work to facilitate construction of the embankment, the geogrid installation and controlled backfilling operations were begun.

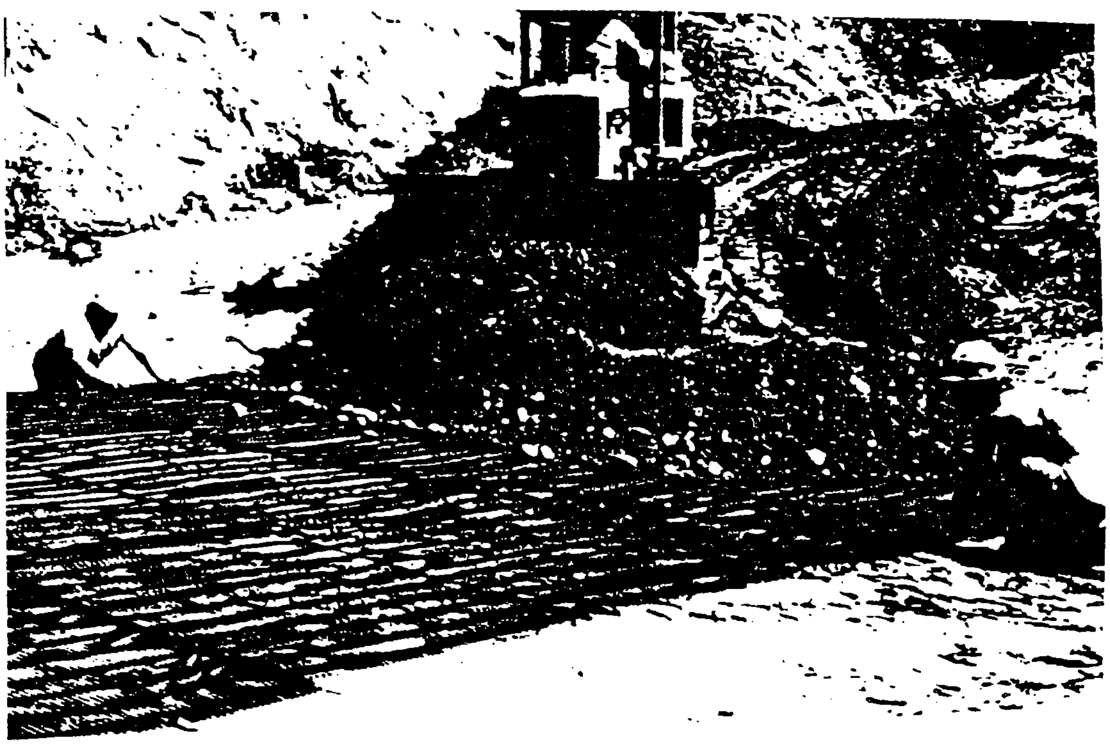
Following are project photos which illustrate each step in the construction process:



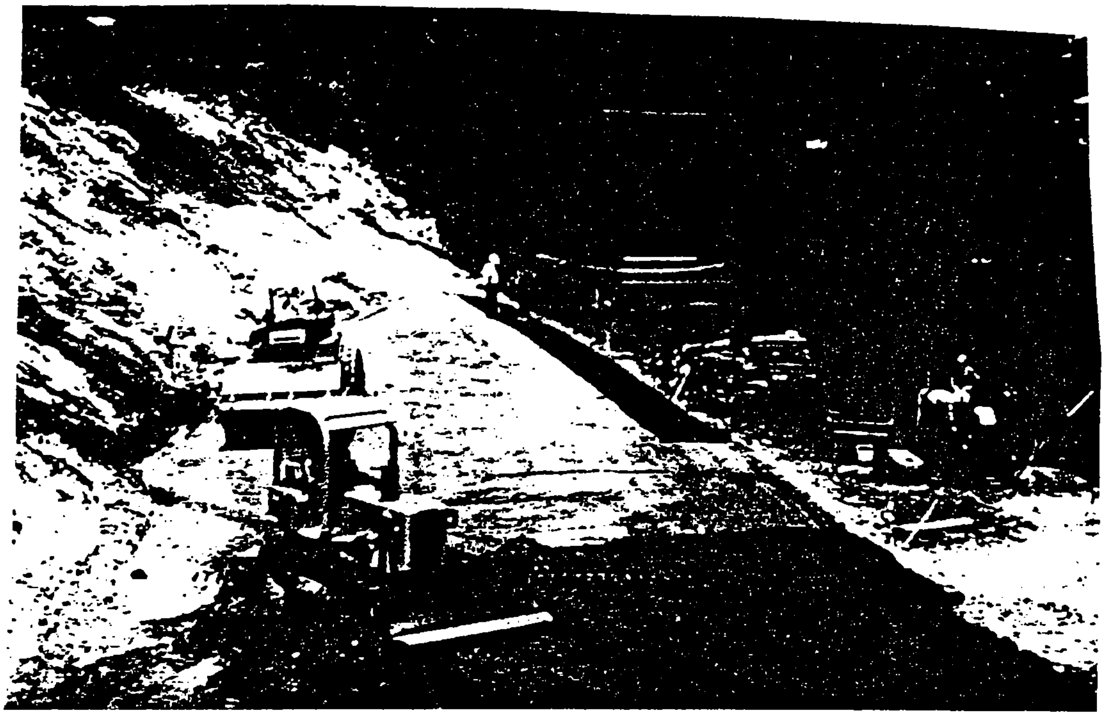
The first step in the process is to roll the material out and cut it to length. This step speeds up construction immensely.



Next, the grids are laid out. Usually a layer of uniaxial grid is the first layer installed.



Backfill is pushed over the grids by a bulldoser.



Then the biaxial material is installed after the next soil lift has been compacted and tested.

This fill, geogrid, fill, geogrid, installation progression continues until the top of the structure has been reached.

Uniaxial geogrids made from HDPE may be positively connected to reduce waste. This is called a "bodkin" connection and has a minimum factor of safety of 2 for the transfer of allowable long-term loads used in the system design.

Note: A layer of top soil was installed at the face as part of the backfill operation. This is so that the topsoil is reinforced. Unrienforced topsoil sitting on the face has a tendency to slide down to the toe of the slope in seeking its natural angle of repose.

The slope was then graded, giving it a smooth, uniform appearance.

Hydroseeding and erosion mat installation completed the construction.

The structure will become part of the natural surroundings within a few years as this photograph, taken in July, 1994 shows.

Conclusion

This was a highly successful project for the New Hampshire Department of Transportation from a number of prospectives. The installed cost of the slope system was 25% less that the cost of a traditional MSE structure while providing the abutter with an aesthetically pleasing slope. This was achieved without sacrificing structural integrity.

The D.O.T. has specified this type of system on other projects and has also specified a mechanically stabilized earth retaining wall using geogrid reinforcement and precast concrete blocks to capitalize on the cost and durability advantages of geogrid-based MSE over traditional steel-based MSE.

Acknowledgments

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Probabilistic Analysis and Design of Slopes on Interstate 26 From Asheville, N.C. to Tennessee

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ABSTRACT

At present, Interstate 26 serves as the corridor between Charleston, South Carolina and Asheville, North Carolina. Proposed is an extension of the interstate from Asheville, north into Tennessee to join Interstate 81 between Kingsport and Johnson City. Although grading in Tennessee is complete to the state line, little construction has been done in North Carolina. In North Carolina, most of the work will consist of upgrading existing four-lane facilities, with a major interchange at Asheville. However, from the small town of Mars Hill to the state line at Sams Gap, about 9.5 miles of major relocation through steep, mountainous terrain will be necessary.

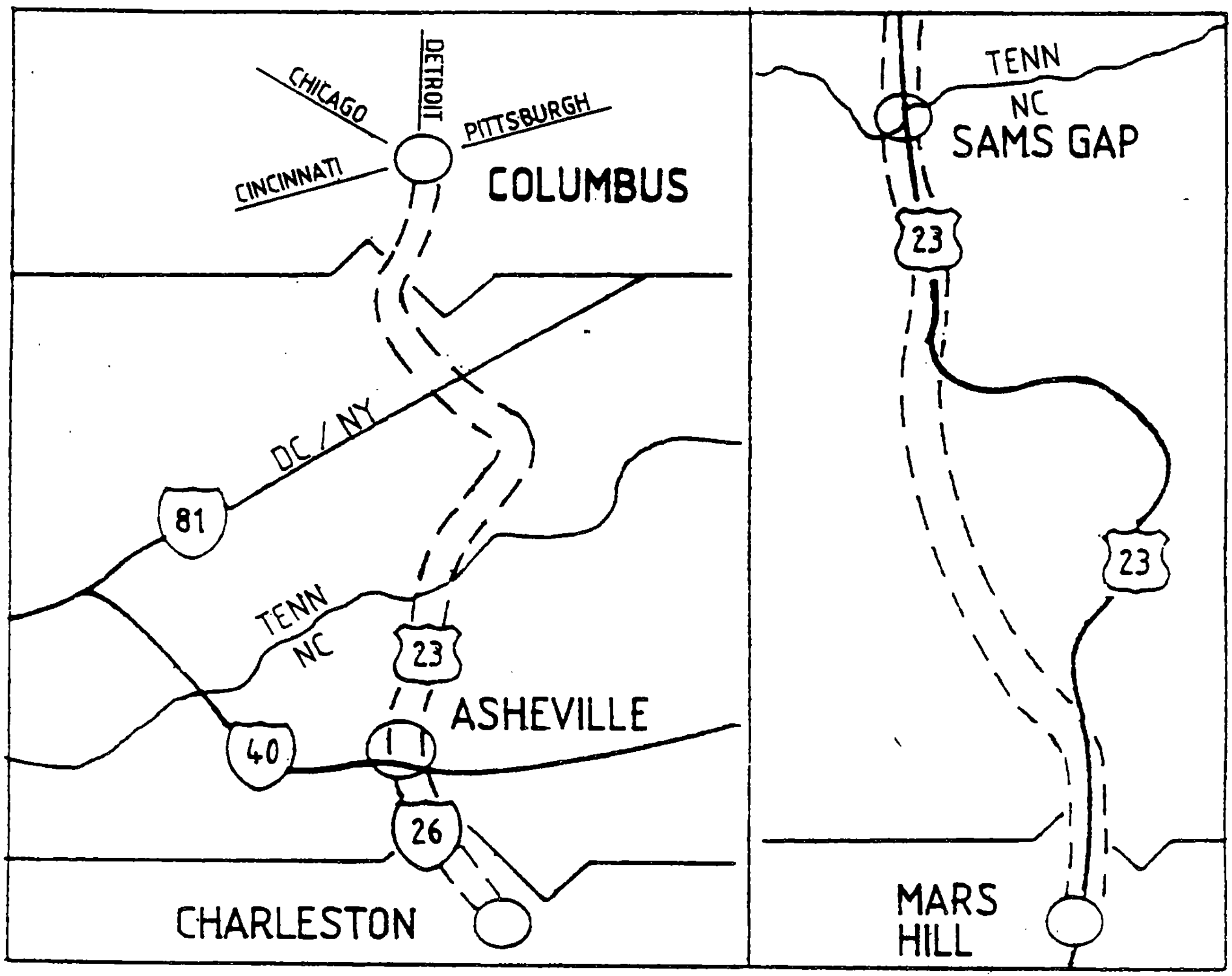
Within this relocated section are at least eleven areas where centerline cuts are 100 to 250 feet, while the maximum height of cuts from the ditchline to the top of the slope can reach up to 500 feet. Fill sections are of a similar magnitude. Included are at least one bridge over 200 feet in height and possibly a series of retaining walls or viaducts used as alternates to large side hill embankments.

Data collection began by detailed mapping and analysis of rock structure within the project corridor. Regional structure was determined by recording the orientation and extent of discontinuities exposed in surrounding road cuts and outcrops in both North Carolina and Tennessee. In ten of the deepest cut sections, additional rock structure was obtained by oriented rock coring methods. Seismic and magnetic surveys were conducted. Strength values were obtained for soil, weathered rock and fresh rock samples, and due to past problems with acidic rocks, samples were tested for acid neutralization potential.

Using these data, the NCDOT has decided to use probabilistic methods of analysis to design the major cuts and fills on this project. The goal of this approach is to arrive at a sound, and especially safe, slope design without being overly conservative due to lack of adequate input data.

Site Description

Interstate 26 is a four-lane facility connecting the coastal areas near Charleston, South Carolina to the Smoky Mountains surrounding Asheville, North Carolina, where it terminates at the heavily traveled Interstate 40. Proposed is an extension of Interstate 26, north from Asheville into Tennessee, joining Interstate 81 between Kingsport and Johnson City, Tennessee. Once completed, this link will provide easier access from both the northeast and the midwestern states to North and South Carolina.



In Tennessee, grading is complete to the North Carolina and Tennessee state line at Sams Gap and the roadway is expected to be open to traffic in early 1995. Once completed, traffic into North Carolina will be funneled onto about eleven miles of a narrow, two-lane road characterized by sharp curves, steep grades and poor visibility.

In North Carolina work will consist of construction of a major interchange at Asheville and upgrading the existing four lanes of US 19-23 from Asheville to north of the small town of Mars

Hill. From this point on to the state line at Sams Gap, about 9.5 miles of major realignment through steep mountainous terrain is necessary to replace the existing steep and narrow roadway which is woefully inadequate for interstate traffic.

Within this section are at least eleven areas where rock cuts vary from 100 to 250 feet in height at the centerline. When combined with excavations in the soil and weathered rock overburden, some of these cuts can approach a maximum height of 500 feet from the ditch line to the top of the slope. Embankment sections are of a similar magnitude and cross steep and marginally stable terrain. A structure of over 200 feet is included and a series of retaining walls to avoid construction of large side hill embankments may be necessary. At one location, a viaduct is being considered to bridge unstable colluvial and ancient landslide slopes.

Regional Geology

The Interstate 26 project is located within the Blue Ridge Belt, a geologic province bounded to the northwest by the Great Smoky thrust fault in Tennessee, and to the southeast by the Brevard Fault zone in North Carolina. Locally, the project runs north through the Walnut Mountains, a Grenville age basement metamorphic complex thrusted unconformably between the Fork Ridge thrust in northwestern Madison County and an unnamed thrust fault to the southeast.

The project area lies entirely within a biotite granitic gneiss which forms a more resistant northeast-southwest ridgeline along the North Carolina and Tennessee state line. This more resistant gneiss creates a relief difference between the beginning of the project realignment in Mars Hill and the steep slopes at Sams Gap. Elevation ranges from 2600 feet above sea level at Mars Hill, to 3800 feet at Sams Gap. Rainfall at Sams Gap averages 60 inches per year, while at Mars Hill, the average is 40 inches per year. The high rainfall has created mass wasting in the form of ancient and historic landslides, many of which are saturated and unstable. Colluvial deposits up to 40 feet in depth are well developed throughout the area along the small stream valleys. High rainfall levels have also created many perched springs and caused rapid erosion by youthful streams.

Probabilistic Analysis

On past projects, the philosophy has generally been to design rock slopes for the worse-case possibility due to the limited availability of geotechnical data and the assumption that if a failure mode is possible, it likely will occur. The result is a very conservative design in which the slope must be designed to eliminate that likelihood. In a project of this magnitude, the North Carolina Department of Transportation realized that using these slope design methods could result in prohibitively costly designs and have a significant impact on the environment.

To address this problem, the NCDOT has committed to the use of probabilistic methods of analysis to aid in the slope design on this project. Probabilism is the doctrine that probability is a necessary and sufficient basis for belief and action, since certainty is unattainable. A probabilistic approach to stability analyses on this project can allow for practically all of the variables in the analysis to be modeled as uncertain variables (Figure 1). These variables and all other uncertainties can then be analyzed and considered to determine which of the uncertain variables are critical to the stability of the slope.

Figure 1. Uncertain Variables (From Keane, 1993)

Geologic Structure

- dip and dip direction
- length, continuity and spacing

Materials Properties

- soil shear strength
- rock compressive strength
- discontinuity shear strength
- soil and rock unit weight

Slope Orientation

- height
- angle
- direction

Tension Cracks

- location
- orientation Groundwater (piezometric) Level Seismic Acceleration

To design rock slopes using this method, a considerable amount of site specific soil and rock data, rock structure orientation and spacing data are required. Preliminary calculations indicated that slopes in rock of 0.5:1 or steeper could result in substantial savings in construction costs. On this project, ten percent of this estimated potential savings has been used to obtain the amount of quality data adequate probabilistic analyses require.

Data Collection

The usual design approach on a project of this nature generally proceeds from data collection through interpretation and analysis to a final design (Figure 2).

Figure 2. Overall Design Approach (From Keane, 1993)

Data Acquisition	Statistical Interpretation	Select Parameters	Reliability Analysis	Design Decision

However, to determine the adequate amount of data necessary to use a probabilistic approach, it is necessary to work backwards through the steps in Figure 2. Considering the needs of the final design requires working back through the analysis, interpretation and data reduction to identify the type, quantity and quality of data necessary. Once identified, it was attempted to utilize the best data collection methods available to measure and describe the uncertain variables listed in Figure 1.

Geologic Mapping

Rock structure data were obtained by mapping outcrops in an area approximately 1000 feet either side of the proposed centerline. Additional data were gathered outside the project corridor to establish the trend and extent of the regional structure.

Approximately 6000 observations were recorded at road cuts and outcrops which included not only the rock type and orientation of the discontinuities, but their length, continuity, infilling materials, waviness, roughness and water conditions. All data were entered into palm top computers in the field using methods and programs developed by Watts and West (1986). The data were then downloaded into a microcomputer where Rockware stereoplotting software established the regional, local and project rock structure. Statistical analyses of the data were done using FRACTAN software. Results of these studies showed a good correlation of rock structure from regional to project scale.

Oriented Core Drilling

Due to the scarcity of outcrops in the project area and the thick vegetative cover, it was decided to obtain oriented rock core in the deepest cut sections. The core would provide statistically valid rock structure data to correlate with the surface mapping and would obtain rock specimens for strength testing directly from the proposed cut slopes. Contracts were awarded to two geotechnical firms for approximately 10,000 feet of drilling to obtain oriented rock core in eleven areas. Of the 10,000 feet, about 650 feet of the drilling was done in angle holes, 45 to 60 degrees from the horizontal designed to insure no near vertical discontinuities were overlooked. The total drilling amounted to 2,425 feet of overburden and 6,791 feet of oriented rock core.

The contracts called for using a multi-shot camera system for rock orientation. In this process, a cylindrical compass and camera unit is mounted on top of an NX wireline inner core barrel. The camera unit is oriented with a set of scribes located in the core catcher which marks the rock as it enters the core barrel. A timing device on the camera allows the driller to stop at any point (in this case, every foot) and take a picture of the oriented compass unit. Depending on the amount of film and drilling conditions, the camera is frequently pulled and opened to develop the film. Development is best done in the field to immediately detect any problem with the camera, development process or drilling technique.

The core is logged with a field description and the percentage of recovery and Rock Quality Designation (RQD) is calculated. Photographs taken in the core barrel are then used to reorient the core in a table top goniometer, a device which holds the core and allows the operator to reposition the core as in-situ. Over 17,000 additional discontinuities were measured in the rock core as if they were in place and the data entered into a microcomputer for analysis. Analysis procedures included separating the data by structure type, plotting poles on stereonets, generating density and contour plots of the poles and picking centroids of groups of poles. Samples of the rock core were then selected to be tested for strength values including unconfined compression, Young's Modulus, Poisson's Ratio and direct shear. The rock was also tested for acid neutralization potential.

Standard Penetration Tests were performed in the soil overburden which was classified and tested for moisture content, recompacted density and triaxial shear. A special effort was made to obtain samples of hard saprolite and weathered rock using dennison samplers and non-orienting core barrels. These samples were tested for direct shear and triaxial shear strength.

Figure 3 is typical of the data collected at each of the deep cut sections.

Figure 3. AREA 7B, Stations 1460+00 to 1475+00

Maximum Slope Height: 380 feet
Drilling: 6 NX oriented core borings

Soil Overburden	66.0 feet
Weathered Rock Overburden	64.0 feet
Rock	928.6 feet
Total	1058.6 feet

Rock Recovery: 91.9% to 100% RQD: 54.6% to 70.4%
12 Unconfined Compression Tests on Rock
  Range: 4,600 psi to 28,800 psi
4 Secant (Young's) Modulus Determinations
  Based on 40% maximum strength
  Range: 4.2 x 106 psi to 7.7 x 106 psi
4 Poisson's Ratio Determinations
  Based on 40% Maximum Strength
  Range: 0.04 to 0.17
4 Direct Shear Tests on Rock
6 Sulfide Content Tests (Acid Neutralization Potential)
  Range NP-AP: -6.50 to 9.93
  420 Discontinuities Recorded in the Rock Core
2 Direct Shear Tests on Weathered Rock
2 Unconfined Compression Tests on Weathered Rock
Standard Penetration Tests in Soil
2 Unconfined Compression Tests on Soil
4 Triaxial Tests on Soil
  Predominant Soil Types: A-4 and A-5
  Average phi': 36.3 degrees
  Average c': 152 psf

Geotechnical Investigation

Between the large cut areas are equally large fill sections which will have to be constructed on steep, side hill slopes. These areas are characterized by an alternating series of small residual ridges and narrow ravines filled with weathered colluvial material. To provide access into these remote areas and into the oriented core drilling sites required construction of several miles of narrow access roads. Construction of these roads required proper permits at each water crossing, no matter how small, and an adequate erosion control program.

The subsurface investigation included rock sounding with solid augers, Standard Penetration Testing through hollow augers, and non-oriented core drilling using NX wireline equipment. Soil samples were tested for quality, moisture content, triaxial shear and recompacted density. The colluvial materials were of special interest, as they are numerous, extensive, deep and usually saturated. The extent of these areas was mapped in detail, as were all obvious springs and wet areas. Consideration has been given to bridging some of these unstable areas by establishing structure foundations in the residual material and leaving the colluvium undisturbed. At other sites, it may be necessary to remove the unstable material and construct a buttress or retaining structure.

Geophysical Investigations

To supplement drilling in the colluvial areas, wide grids were laid out for detailed seismic and magnetic surveys to determine the nature of residual material and the depth and extent of colluvium. Seismic data were analyzed in part using the Generalized Reciprocal Method which takes information from overlapped seismic lines and yields a continuous refraction profile. The seismic information, when correlated with boreholes in the area, gave a reasonably accurate estimate of the colluvial materials.

Magnetometer surveys were conducted using a portable, hand held electron precession magnetometer. Rock of similar lithology and of similar tectonic origin yields a magnetic signature which remains fairly consistent throughout a local area of homogeneous rocks. Materials that have formed landslide and colluvial deposits tend to cancel out the magnetic signature due to the chaotic nature of the deposit. When contoured, the magnetic data will show strong values for rock in place and increasingly weaker values when the instrument passed into a colluvial deposit. This contrast helps determine the extent of a deposit despite weathering or other factors which may obscure its boundaries.

Other areas on the project appear on the ground surface to be slump features containing no visible boulders or rock fragments. Data from seismic and magnetic surveys, as well as boreholes, show that although the material is similar to colluvium, these features are more likely to be ancient landslides. Slope inclinometers have been installed in these areas to record any movement.

Design Decisions

The structural and strength data obtained at each site can be statistically evaluated to determine probability distributions which describe the uncertainty of the variables. The resulting statistics are used in analyses determining the probability of failure along circular, planar and wedge modes, toppling and rockfall using Reliability Methods and Monte Carlo simulation. Results include the probability of failure and the expected volume of the failure mass.

Although not addressed in this paper, much the same procedures are being used to design cut slopes in both soil and weathered rock. Equally important is the design of fill slopes which must be constructed on the steep and sometimes marginally stable natural slopes.

The design procedures used on this project are the result of the combined efforts of engineering geologists, geotechnical engineers and academic specialists. The entire process is being reviewed by acknowledged experts in probabilistic rock slope analysis. The expected results are to select the optimum slopes with respect to construction costs, construction risk, programmed maintenance, maintenance risk, public safety and aesthetics.

References

Keane, J.M. (1993). Site Characterization for Probabilistic Analysis and Design of Highway Slopes. ASCE Geotechnical Special Publication No. 37, 102-113.

Watts, C.F. and West, T.R. (1986). Discontinuity Significance Index and Electronic Data Collection for Rock Slope Stability Studies. Bulletin of the Association of Engineering Geologists, 23-3, 265-278.

Effects of Lime on Cannon Creek Embankment Soil

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ABSTRACT

Cannon creek is the site of a 76 feet high by 800 feet long geogrid reinforced embankment built with an expansive clay. The clay is classified AASHTO A-7-6 (37) with an average liquid limit of 64 and a plastic index of 37. Even though the embankment with side slopes of 26.5° has performed well since its construction in 1988, the backslope of 11.3° where fill was obtained has failed. The failed section is 400 feet long by 250 feet wide.

The objective of this project was to determine the effect of lime on the expansive clay properties. According to the results obtained from the laboratory testing, the angle of internal friction increased from 4° for the in situ soil to 24° for 6% lime content cured at 50 days. The percent of clay size soil particles were reduced by a factor of 4. The plastic index for the in situ soil was reduced from 37 to 20 with the addition of 2% lime. Unconfined compression strength of the lime treated soil increased with curing time for lime contents 4 through 10%. Six percent lime treated soil has an unconfined compression strength of 175 psi after 50 days.

INTRODUCTION

A slope stability failure has developed at Cannon Creek which is located on Highway 16 east of Fayetteville, Arkansas. It is the site of a 76 feet high by 800 feet long geogrid reinforced embankment built with an expansive clay. The slope failure has developed in the borrow area from where the fill was obtained. The slope failure first developed in 1989 (Figure 1; McGuire, 1990) and is now (1994) approximately 400 feet long by 250 feet wide (Figure 2). Since its construction in 1988, the geogrid reinforced embankment has performed well with 26.5° side slopes. The failed borrow area, however, has a slope of only 11.3°.

BACKGROUND

The objective of this project is to identify the effect of lime on the soil properties. Lime increases the grain size and unconfined compression strength of clay. In addition lime reduces the plastic index of clay soils (Transportation Research Circular No. 180, 1976; Ingles and Metcalf, 1973; and Hausmann, 1990). Even though friction angles of 25° to 35° (Thompson, 1966) and 19° to 38° (Tuncer and Basma, 1991) have been reported, the effect of lime on the angle of internal friction is not well known.

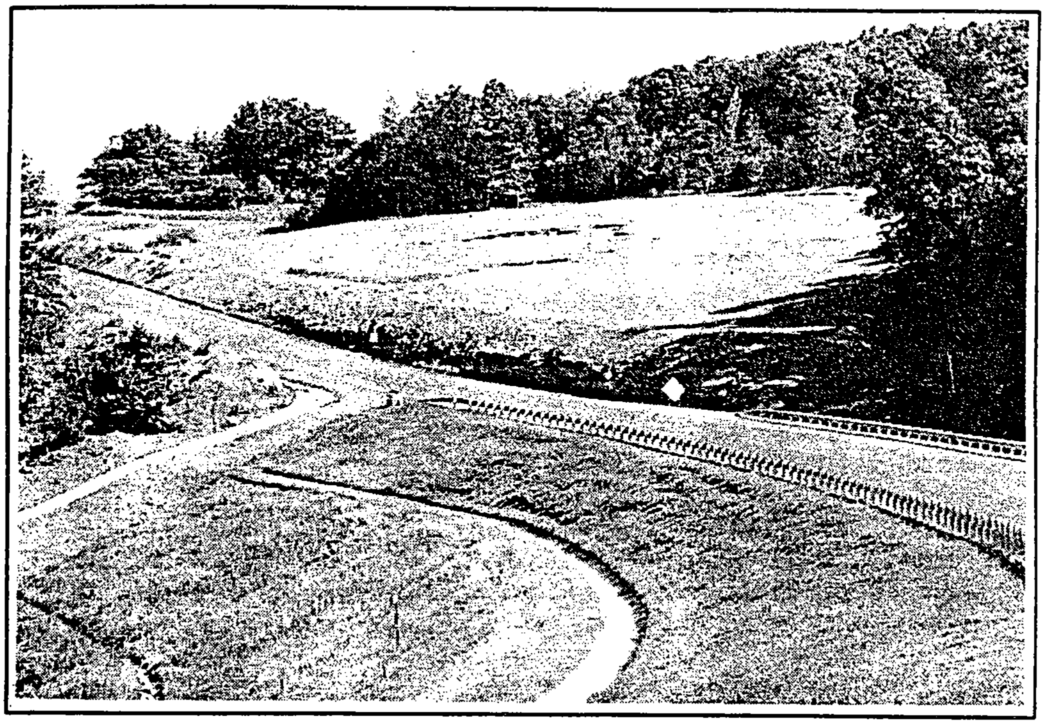

FIGURE 1. Initial Slope Failure in July, 1989 (McGuire, 1990).

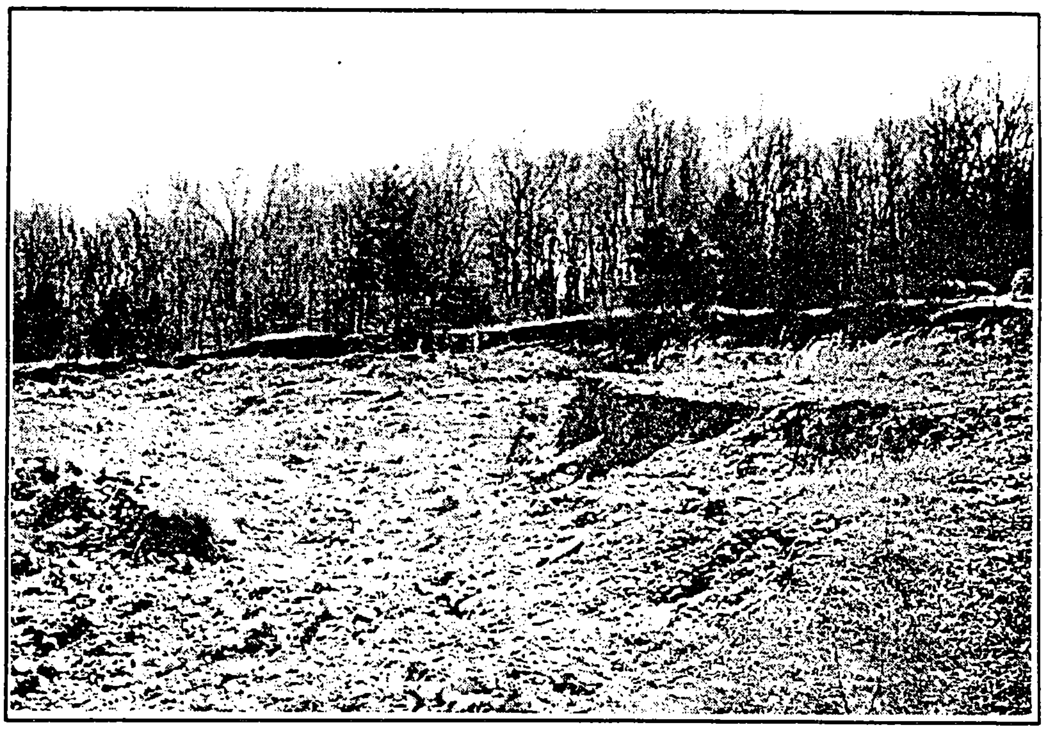

FIGURE 2. Slope Failure in April, 1994.

Cannon Creek embankment soil was selected for the study because the soil was used in a Federal Highway demonstration project and was previously published in the 1990 and 1992 Highway Geology Symposiums. "Geogrid-Expansive Clay Embankment", by Thornton and McGuire, 1990, was the first report and "Cannon Creek Embankment Instrumentation", by Thornton, McGuire and Thian, 1992, was the second report.

FIELD OBSERVATION

By March 1994, the soil at the heel of the failure plane has declined approximately 7 feet in elevation since the slide started to fail in 1989 (Figure 3). At the toe of the slide, the soil mass pushed over a traffic sign and is filling the concrete ditch (Figure 4). The accumulated soil may divert the water flow into the roadway during rainfall which could be hazardous to traffic operations. This requires the Arkansas State Highway and Transportation Department to periodically clean out the ditch.

LABORATORY INVESTIGATION

Lime contents of 2, 4, 6, 8, and 10% were used in the laboratory investigation. Triaxial samples were prepared in compaction molds using standard compaction effort and cured for 7, 28, and 50 days.

The following tests were conducted to determine the changes in engineering properties of the lime treated soil:

Multistage Triaxial
Unconfined Compression
Liquid Limit
Plastic Limit
Hydrometer
Standard Proctor Compaction
Consolidation (for compressibility)
X-Ray Diffraction

RESULTS

Multistage Triaxial Test

Angle of Internal Friction

Longer curing periods and higher lime contents improved the internal friction of the treated soil. The untreated soil has an angle of internal friction of 4°; the soil treated with 6% lime content has an angle of internal friction of 15° after 7 days, 16° after 28 days, and 24° after

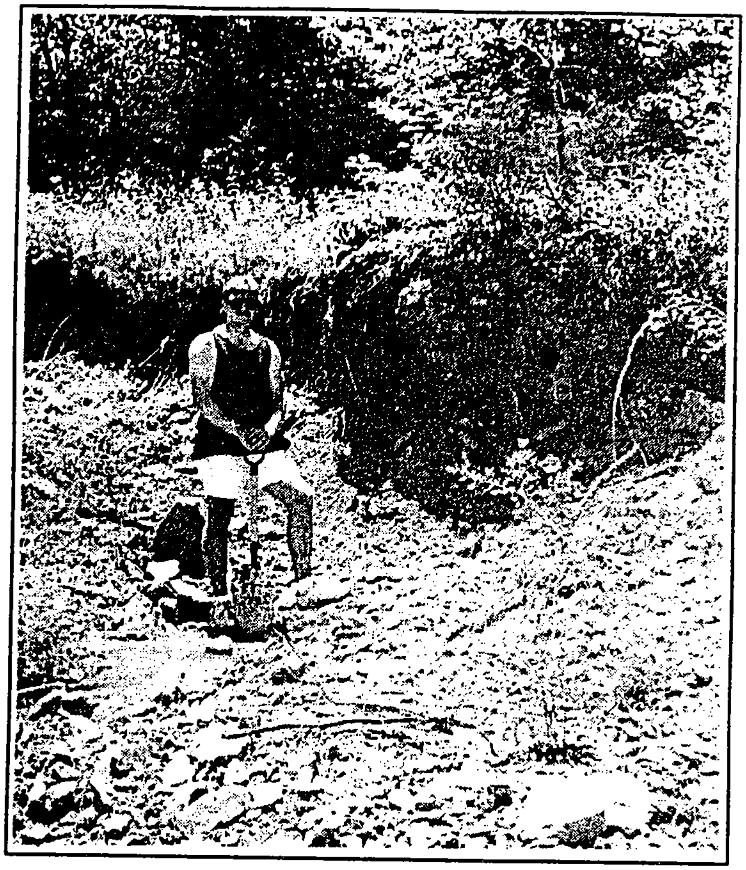

FIGURE 3. Heel of Slope Failure.

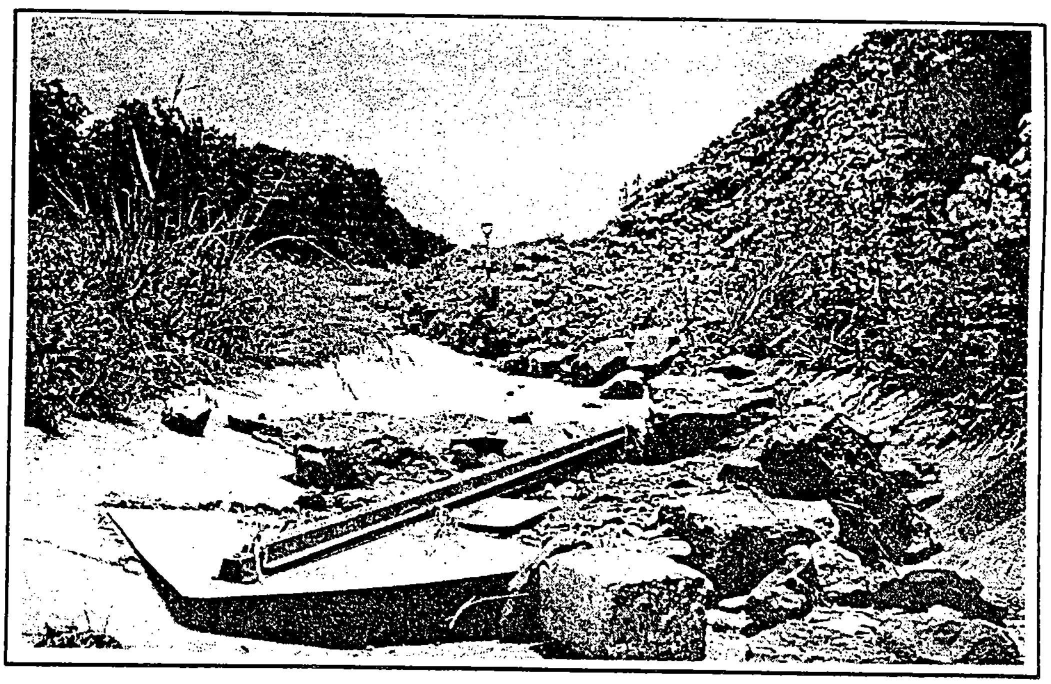

FIGURE 4. Filled Ditch and Road Sign.

50 days (Figure 5).

The 7, 28, and 50 day lines in Figure 5 are the least square best fit for internal friction as determined from Mohr's circles. Table 1 contains the averaged triaxial data used to construct Figure 5. An example of the internal friction determination from the Mohr's circles is illustrated in Figure 6.

TABLE 1. Average Angle of Internal Friction.

	Lime Content by Weight				
Curing Times	2%	4%	6%	8%	10%
7 Days	4°	10°	10°	26°	25°
28 Days	3°	5°	23°	25°	20°
50 Days	17°	19°	14°	38°	33°

Cohesion

Cohesion intercepts were associated with the angle of internal friction of Table 1 (Figure 6). There is no significant increase in cohesion with respect to time for 2% lime content. However, for 4 to 10% lime samples, the cohesion increased with time but not lime content. The cohesion has an average value of 24 psi at 7 days, 40 psi at 28 days, and 48 psi for 50 days.

Unconfined Compression Test

The unconfined compression strength increased with time for lime contents 4, 6, 8, and 10% (Figure 7). The strength for the in situ soil was 6 psi. with the addition of 6% lime, the strength increased to 50 psi at 7 days, 110 psi at 28 days, an 175 psi at 50 days.

The gain in strength for a given lime content is primarily due to the increased curing time. A lime content above 6% has little effect on the unconfined compression strength of the soil.

Liquid Limit and Plastic Index

The liquid limit was reduced from 64 for the in situ soil to an average of 58 for treated soils (Figure 8). The plastic index was reduced from 37 for in situ soil to 20 for soil with 2% lime. The plastic index decreased slightly as lime content increased above 2%.

Grain Size

Treatment with 2% lime only slightly increased the grain size of the clay particles as determined by hydrometer analysis. However, the clay fraction is reduced from 72% to 18% by weight of soil particles for the soil with 6% and 10% lime (Figure 9).

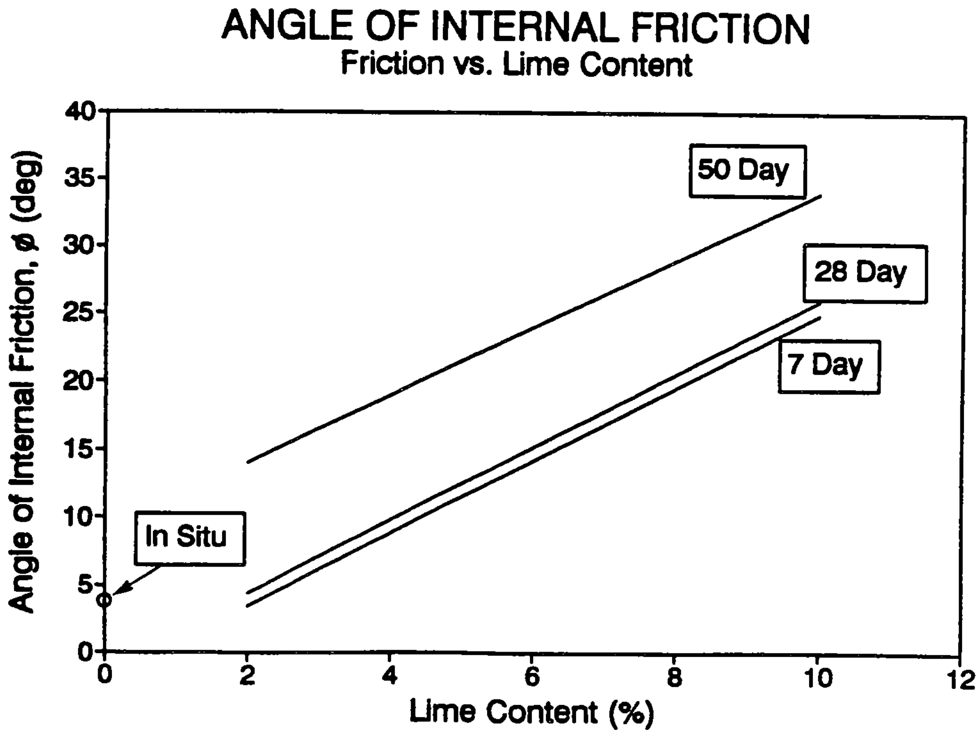

FIGURE 5. Angle of Internal Friction.

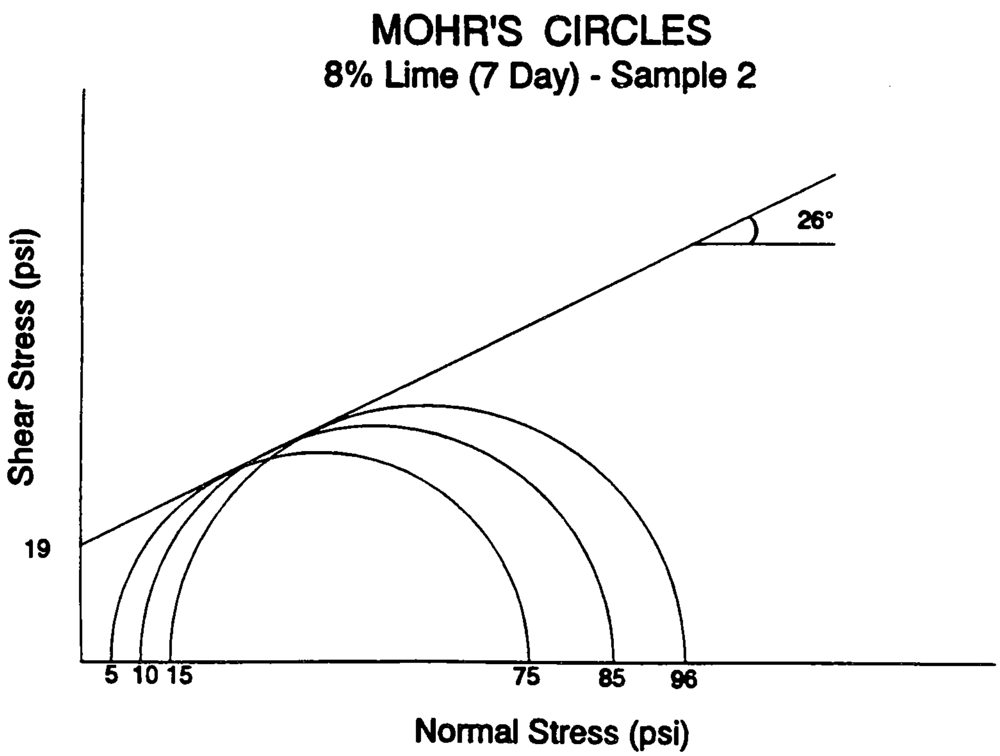

FIGURE 6. Mohr's Circles.

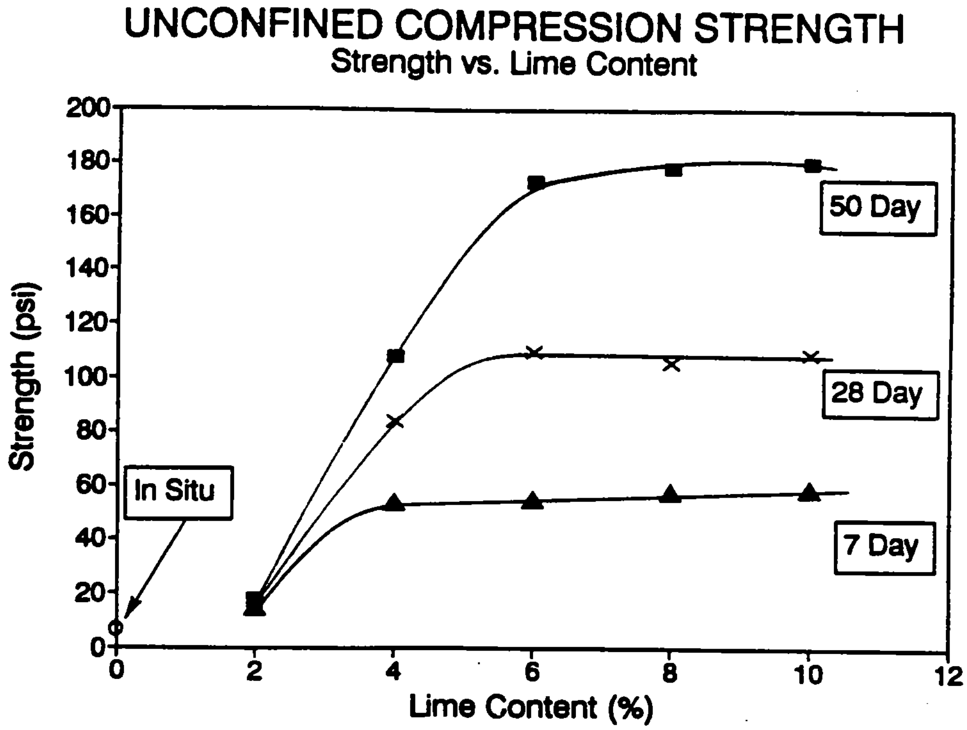

FIGURE 7. Unconfined Compressive Strength.

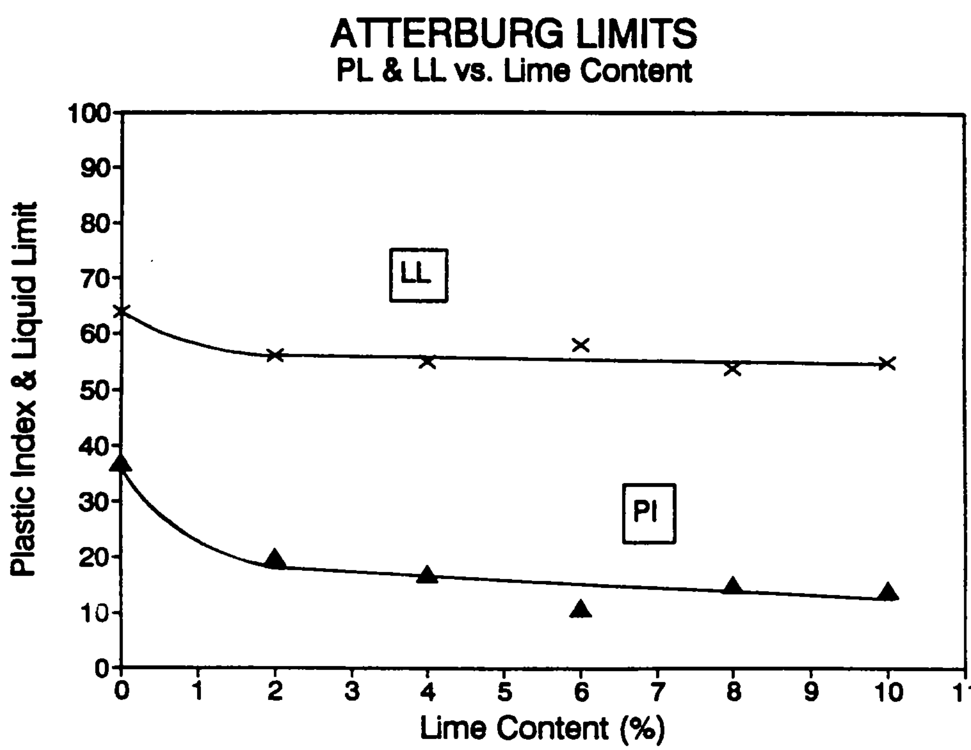

FIGURE 8. Atterburg Limits.

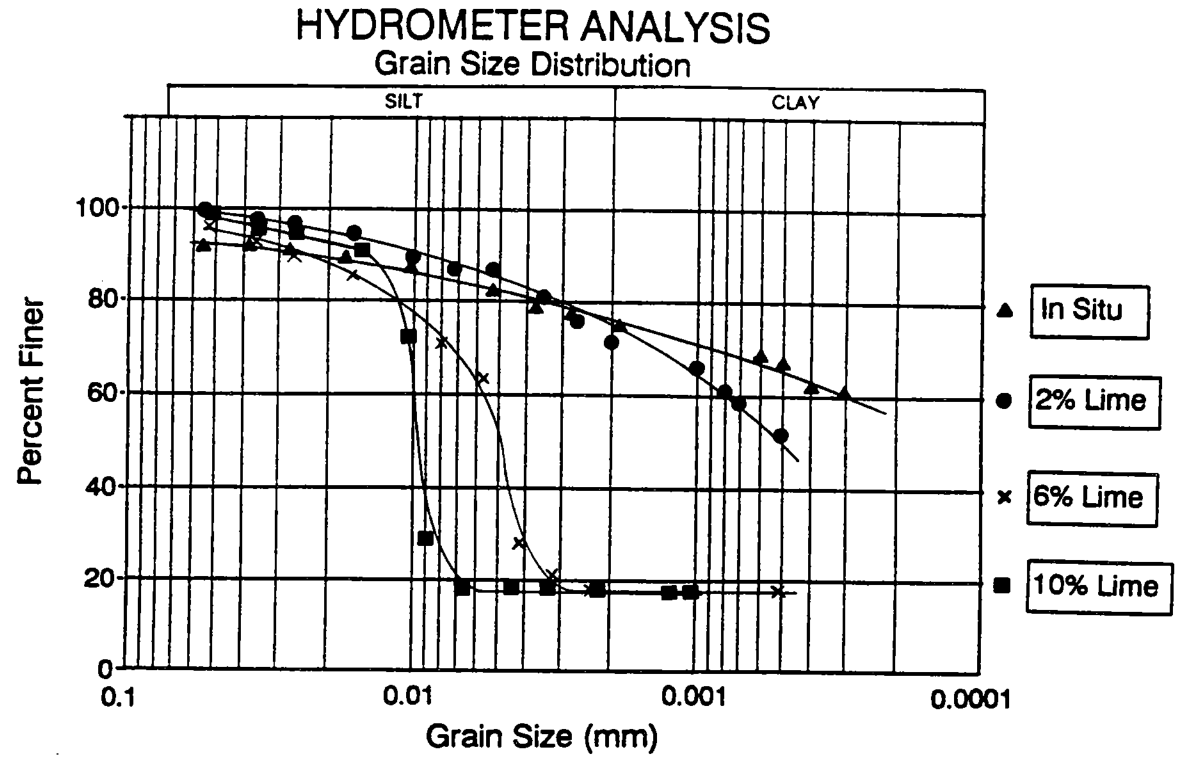

FIGURE 9. Hydrometer Analysis.

Standard Proctor Compaction Test

The addition of lime significantly increased the optimum moisture content and decreased the maximum dry unit weight. The optimum moisture content for the in situ soil was 24%, and the optimum moisture content for the 6% lime treated soil was 30%. The maximum dry unit weight decreased from 94 lb/ft^3 for the in situ soil to 90 lb/ft^3 for the soil containing 6% lime (Figure 10).

Compressibility

From the consolidation test, the compressibility of the 6% treated soil was decreased by a factor of 20 compared to the in situ soil. The coefficient of consolidation C_c, was 0.32 for the in situ soil and 0.016 for the soil treated with 6% lime cured for 47 days.

X-Ray Diffraction

The major clay minerals in the expansive clay, determined from the x-ray diffraction test included montmorillonite or a mixture of montmorillonite and vermiculite, chlorite, kaolinite, and quartz.

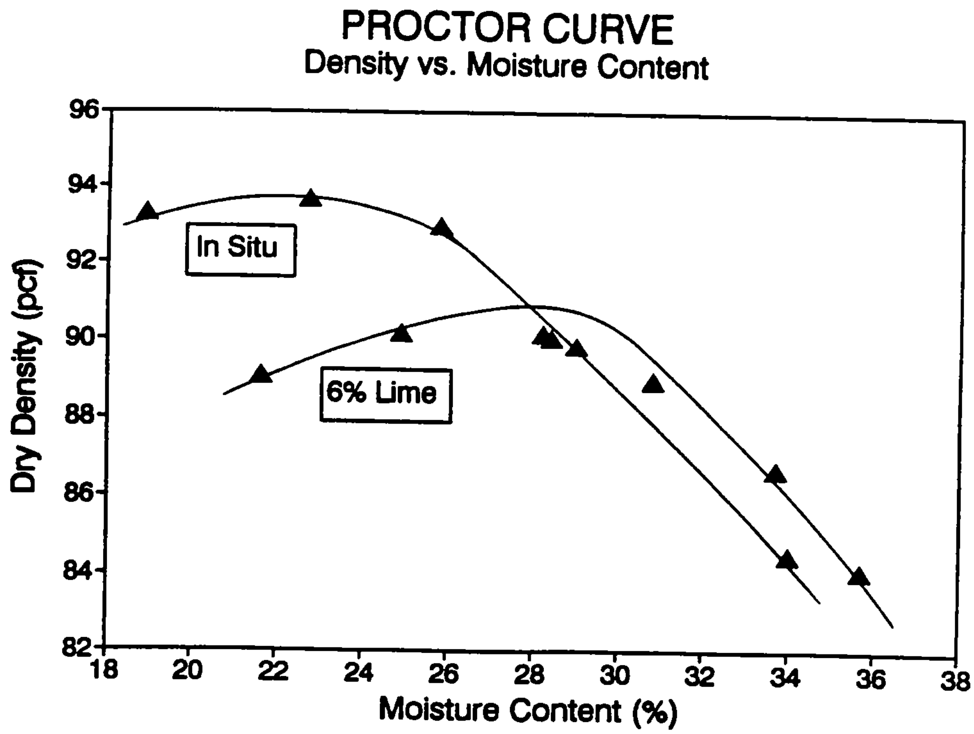

FIGURE 10. Proctor Curve.

FINDINGS

1. The angle of internal friction increased with increasing lime content and curing time.
2. Lime increased the grain size of the clay size soil particles.
3. The plastic index for the in situ soil was reduced by approximately 50% when 6% lime was added.
4. For 6, 8, and 10% lime content, the unconfined compressive strength depended on curing time.

REFERENCES

Haussmann, Manfred R., 1990, *Engineering Principles of Ground Modification*, McGraw-Hill, Inc.

Ingles, O. G., and Metcalf, J. B., 1973, *Soil Stabilization*, John Wiley and Sons, New York, NY.

McGuire, Michael S., 1990, "Reinforced Earth Embankment", Master's Thesis, University of Arkansas.

Thompson, M. R., 1966, "Shear Strength and Elastic Properties of Lime-Soil Mixtures", Highway Research Record 139, HRB National Research Council, pp 1-14.

Thornton, Sam I., McGuire Michael S., and Thian, Boon K., 1993, "Cannon Creek Embankment Instrumentation", 43rd Annual Highway Geology Symposium, Fayetteville, AR.

Thornton, Sam I. and McGuire, Michael S., 1990, "Geogrid-Expansive Clay Embankment", Proceedings of 41st Annual Highway Geology Symposium, Albuquerque, NM.

Transportation Research Circular No. 180, 1976, "State of the Art: Lime Stabilization", Transportation Research Board, National Academy of Sciences, Washington D. C.

Tuncer, Erdil R. and Adnan A. Basma, 1991, "Strength Characteristics of a Lime-Treated Cohesive Soil", Transportation Research Record 1295, TRB, National Research Council, p 71.

INNOVATIVE ROCK ANCHORING AT BOUNDARY DAM, WASHINGTON

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ABSTRACT

High capacity post-tensioned rock anchors were selected by Seattle City Light to stabilize a critical rock abutment, supporting Spillway No. 1 and the Disposal Channel at Boundary Dam, against sliding and potential seismic loadings. Drilling and installation of the anchors posed unique challenges from a design and construction standpoint. The paper addresses the geologic conditions, design, construction and performance of the 1,000 kip and 500 kip anchorages installed from the dam access tunnel through the unstable rock mass daylighting at the exterior rock face. Key aspects of the anchor installation including inclined directional drilling, consolidation grouting, use of doubled-headed anchors, and construction monitoring are also discussed.

INTRODUCTION

Boundary Dam, a variable radius double curvature concrete arch dam constructed between 1963 and 1967, is located on the Pend Oreille River in the northeastern corner of Washington State, approximately one mile south of the US-Canadian border (Figure 1). The dam stands 360 feet above the deepest part of its foundation and has a crest length of 508 feet (740 feet including the spillways). It is a major feature of The Boundary Hydroelectric Project supplying 1,000 MW of power to the Seattle metropolitan area 350 miles away.

In 1990, a review board was formed by Seattle City Light (SCL), at the request of FERC to investigate the safety and stability of the dam. One area of concern identified by the board for further investigation was the stability of two outer wedges of rock located in the cliff at the dam's left abutment, which was formed by a system of joints and faults (Figure 2). The dam is situated in a deep canyon with steeply sloping abutments on either side. The rock on which the dam is founded is interbedded limestone and dolomite, which is relatively hard but has wide variations in physical characteristics. This rock provides support for the spillway chute, as well as providing confinement for the dam foundation which carries the arch loads. It was determined that the rock wedges were potentially unstable in sliding as well as during seismic loading and an application of an anchoring load equal to approximately ten percent of the dead weight of the wedges would be required to improve the static factor of safety and reduce the potential for dynamic displacement during seismic loading.

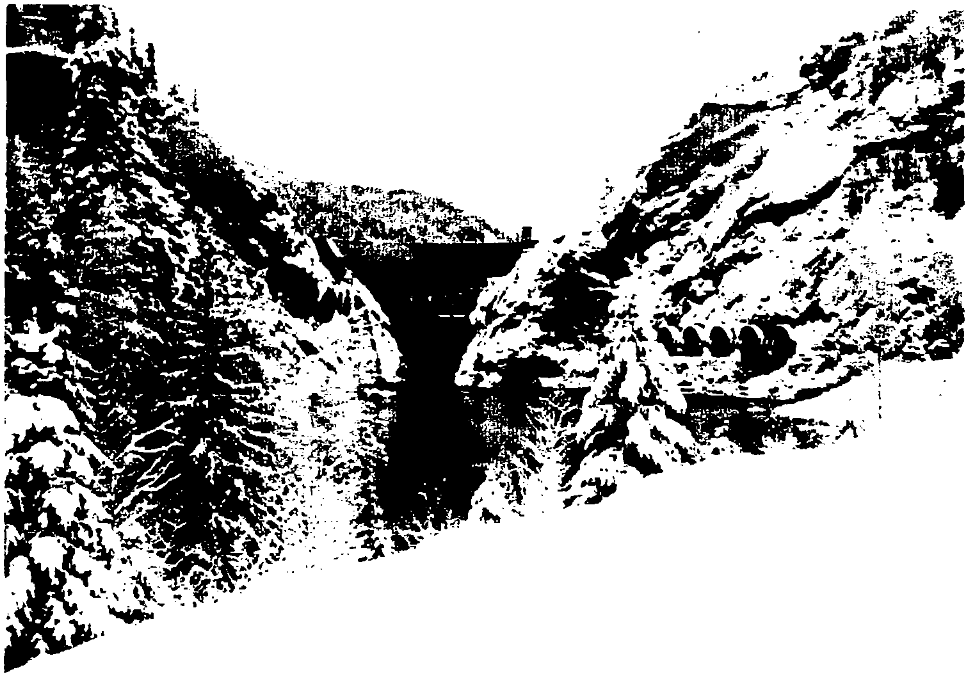


Figure 1 (Above)

Downstream View of Boundary Dam

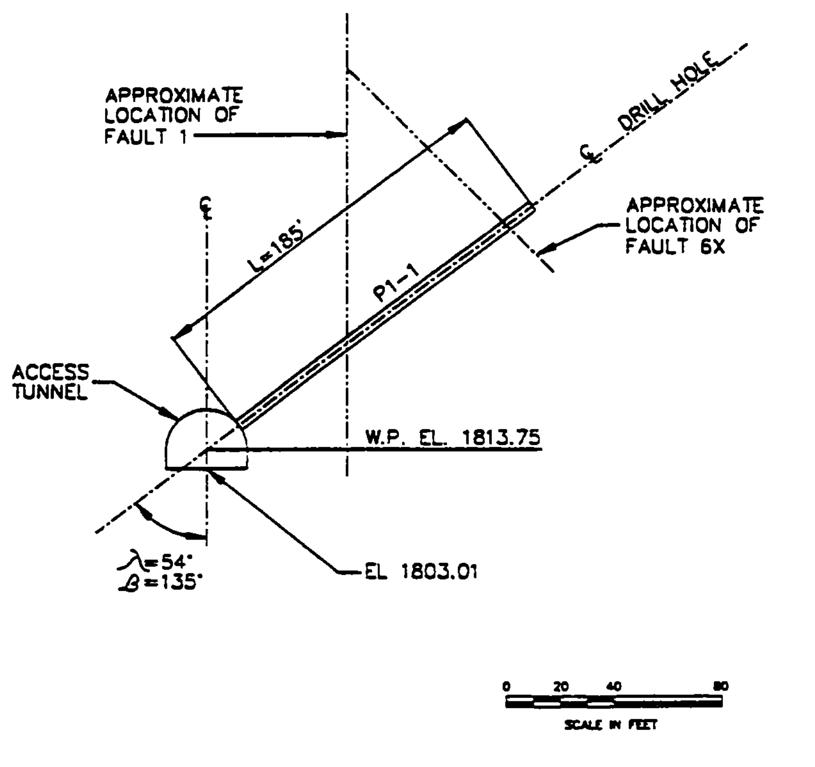
Figure 2 (Right)

View of Rock Wedge at Left Abutment from Dam Face

ANCHOR DESIGN

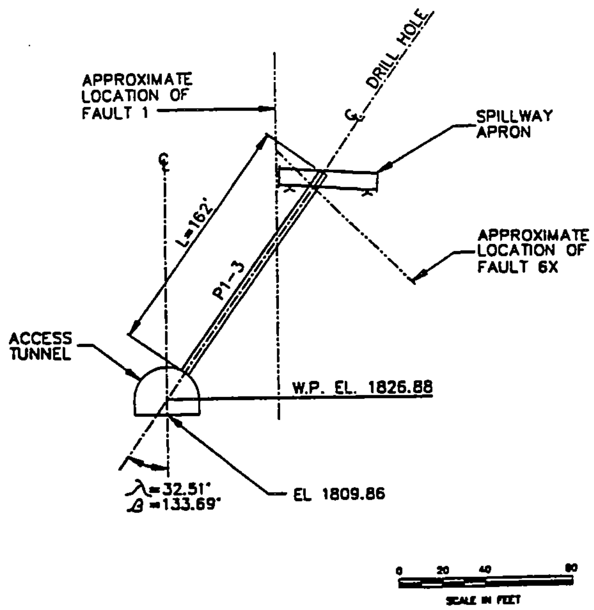
Initial concepts for the anchor scheme considered either installing the anchors from the cliff face and founded into the rock below the two fault lines or installing the anchors up from the dam gallery access tunnel which runs through the cliff behind the rock wedges, these anchors being founded in the wedges themselves.

The final design completed by Harza Northwest, Inc. Engineers included installation of six each 1,000 kip capacity anchors and seven each 500 kip capacity anchors. All of the anchors were to be installed from the gallery access tunnel. All of the anchors had an upward inclination, with alignment angles to the horizontal ranging from 18 to 58 degrees (Figures 3, 4 and 5). The 1,000 kip design capacity anchors were designed as double-headed anchors (no bond length) due to the end of the anchors being located on or adjacent to the spillway channels, allowing relatively easy access for installation of the outside end bearing plate and anchor head. The 500 kip anchors would exit the rock on the face of the cliff, and were terminated and bonded in the rock wedges in order to avoid work on the cliff face. Total anchor length ranged from 89 feet to 207 feet. The tendons comprised 0.6" diameter stands, 31 for the 1,000 kip anchors, and 16 for the 500 kip anchors.



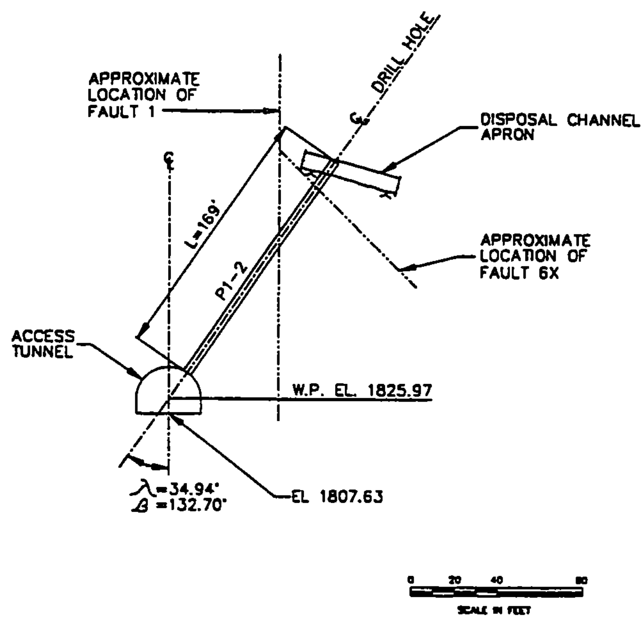
SECTION THRU ℄ P1-1 ANCHOR HOLE

FIGURE 3. SCHEMATIC REPRESENTATION OF PRESTRESSED ANCHORS EXITING ROCK FACE



SECTION THRU ℄ P1-3 ANCHOR HOLE

FIGURE 4. SCHEMATIC REPRESENTATION OF PRESTRESSED ANCHORS EXITING AT SPILLWAY APRON.



SECTION THRU ℄ P1-2 ANCHOR HOLE

FIGURE 5. SCHEMATIC REPRESENTATION OF PRESTRESSED ANCHORS EXITING AT DISPOSAL CHANNEL APRON.

CONSTRUCTION

Drilling, water testing, and installation of the anchors posed some unique challenges. The anchors were installed by drilling inclined holes of varying inclinations above horizontal from within the dam access tunnel in the left abutment area. The original design allowed bonded anchors when sufficient depth of sound rock existed beyond Fault 6X. Nicholson Construction Company (Contractor) elected to daylight all the drill holes at the exterior rock face (Figure 6). This procedure allowed for the installation of all anchors as double-headed anchors. The use of double-headed anchors improved anchor quality as well as constructability, eliminating the need to provide adequate bond of the anchor into the rock wedge with a mechanical connection. Rock climbers were employed to gain access to the drill hole exit points on the rock face to perform associated work including excavation of recesses for the exterior anchor head assemblies, tendon installation, installation of exterior anchor head assemblies and exterior blockout restoration.

Figure 6: Drill Hammer Exiting Drill Hole

Drilling of the anchor holes was completed using a 10-inch diameter down the hole hammer advanced with a Davey Kent DK-90 rotary drill rig. To maintain proper alignment of the drill hole, the drill mast was removed from the drill chassis and mounted on a rigid frame, with the upper end of the mast bolted to the wall of the 24-foot high tunnel at the surveyed anchor point of entry (Figure 7). The alignment criterion for the 1,000 kip capacity anchors exiting in the spillway was one-half inch per 10 feet of anchor length. Drill hole alignments were verified in the field by instrument survey and utilization of a digitilt inclinometer in the drill hole. Results of all of the alignment tests, performed after initial drilling, and again after consolidation grouting and redrilling, fell within the specified criteria. A 26-foot long composite rigid stabilizer pipe on which the percussive hammer was mounted allowed further control of the anchor alignment.

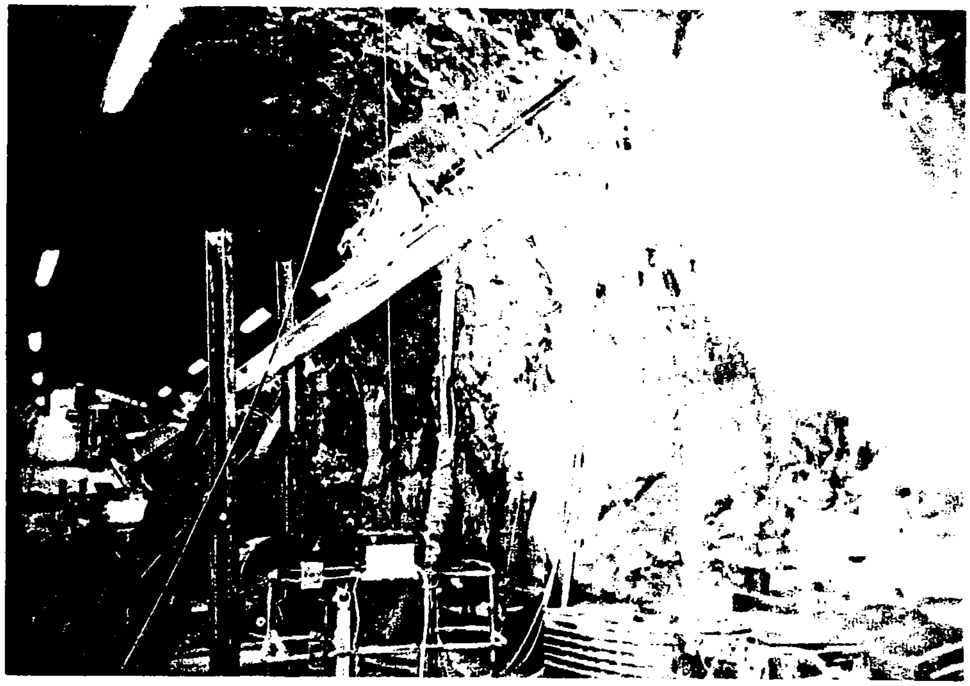


Figure 7: Drill Set-up Inside Access Tunnel

The length of the completed drill hole and the location of the Fault Line 6X were inspected using a video camera. The holes were then tested for water tightness. The testing showed excessive water loss in all holes. The required procedure was to plug the bottom of the hole with a packer, fill the hole with water, seal the top of the hole with a packer, then induce water at a pressure of five psi while measuring the flow rate for a 10-minute period. The specified maximum flow rate was 0.01 gallons per linear foot of hole per inch of hole diameter. This initial water test was completed on only one of the holes, which did not meet the criteria. The remaining holes would not fill up with water, which was pumped in at a rate exceeding fifty gallons per minute. In order to provide a grout tight hole, which allows more reliable grouting and proper corrosion protection of the tendons, the holes were consolidation grouted and redrilled. Water testing performed after redrilling showed results less than the specified maximum flow rate.

The rock at each end of the anchor was chiseled level to provide a bearing surface for the 18-inch to 30-inch diameter bearing plates. Bolts were installed on which the plates were mounted and aligned. The void under the plate was filled with nonshrink grout. The tendons were inserted into the tunnel end of the anchors and hoisted up with a pulley and cable system hooked to an air-powered winch located on the spillway apron above. The full lengths of the tendon strands were individually covered with a grease filled plastic sheath.

As the anchor connections were purely mechanical, conventional anchor performance testing was not required. The required lockoff load (1.2 x design load) was applied individually to each strand using a single strand jack. Elastic elongations of the strands were measured and compared to the theoretical value, and liftoff testing of the strands was performed to verify proper loading of the tendons.

Upon acceptance of the anchor loading, secondary grouting of the tendons and blockout restoration were completed. For all anchors, the stressing heads at the tunnel end anchorages were provided with painted steel caps to protect the stressing heads, wedges and strands (Figure 8). The blockouts at the exit points beneath the spillway aprons were restored by replacing the disturbed slab reinforcement and backfilling with concrete to match the adjacent apron surfaces (Figure 9). For the remaining exterior blockouts on the cliff face, grout backfill was placed to complete blockout restoration (Figure 10).

Existing drain holes located in the vicinity of the anchors were previously installed to relieve hydrostatic pressure in the faults and fracture zones in the left abutment. During consolidation grouting of the anchor drill holes, these existing drain holes were sealed due to the interconnection of the drains and drill holes via Fault Line 1.

Five new drain holes were drilled to replace the sealed existing drain holes noted above. Each drain hole was drilled to intercept the fault 6X plane. Depths were determined based on drill hole video camera inspection data and previous drain hole data.

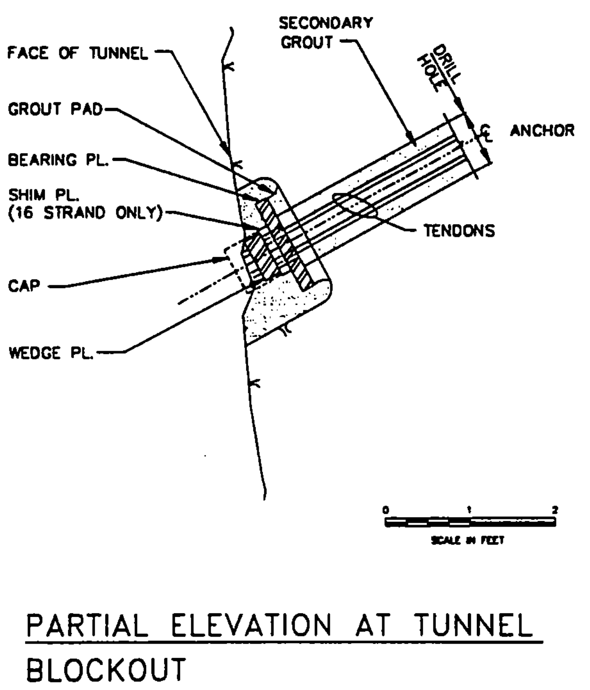


FIGURE 8. ANCHOR HEAD/RECESS DETAIL AT DAM ACCESS TUNNEL

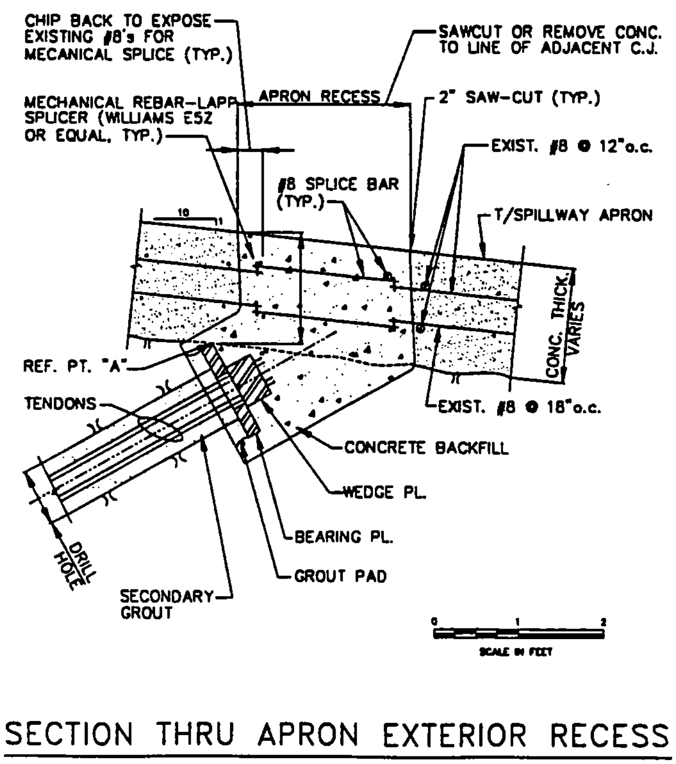
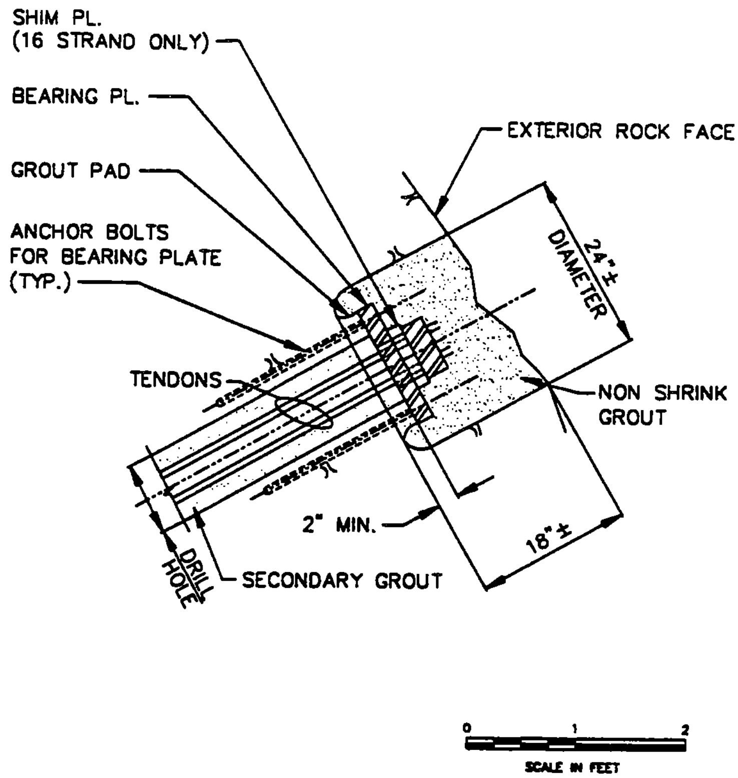


FIGURE 9. ANCHOR HEAD/RECESS DETAIL AT SPILLWAY AND DISPOSAL CHANNEL APRONS.



SECTION THRU EXTERIOR RECESS
(P2-SERIES ANCHORS)

FIGURE 10. ANCHOR HEAD/RECESS DETAIL AT ROCK FACE.

UNIQUE PROJECT FEATURES

Three full-time rock climbers were used for completion of the work on the cliff face 200 feet above the valley floor (Figure 11). Equipment for this work was limited to hand-held tools.

The difficulty of the climber's work was greatly increased by the extreme weather experienced that winter. The project commenced October 1992 and was completed at the beginning of April 1993. Near record snowfall and wind chill temperatures as low as 20 degrees below zero were experienced. Ice buildup on the cliff face from water flow off the spillway reached thicknesses of four feet. Warm air flowing up the anchor shafts helped keep the ice back from the exterior recesses. Despite all the difficulties of access and weather, the project was completed without experiencing a lost time accident.

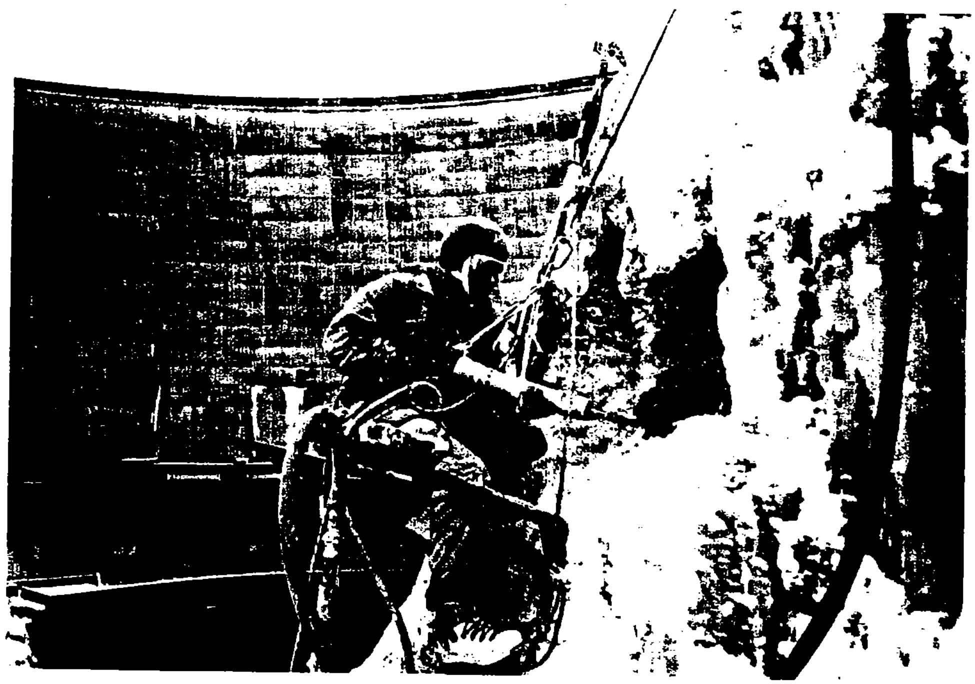


Figure 11. Rock Climber at Cliff Face
Hand Digging Anchor Recess

CONCLUSIONS

The completed Boundary Dam Left Abutment Foundation Anchor System provides additional sliding stability for the rock wedge formed by the intersection of Faults 1 and 6X. Sliding Safety Factors of 1.4 and 1.1 have been achieved for the dead load and seismic load cases. Innovative construction techniques provided by the contractor improved construction scheduling and long-term anchor performance properties.

REFERENCES

Bruce, D.A. and Groneck, P.B. (1993), "Dam Repair by Anchoring and Grouting: Some Recent Notable Case Histories".

Harza Northwest, Inc. (July 1992), "Design Basis Report, Foundation Anchors at Boundary Dam".

Harza Northwest, Inc. (May 1993), "Foundation Anchors at Boundary Dam—Final Report"

Rock Excavations in Steep Ground

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ABSTRACT

Several recent highway projects in Washington State have provided valuable experience in the project scoping, design, contract development, and construction management of rock slopes located in steep ground. High construction costs and the post construction stability and safety of these slopes require geotechnical input during all phases of project development for the evaluation of design alternatives, review of design objectives, and constructibility. The geotechnical design should consider, at a minimum, the presence and extent of overburden and highly weathered rock, zones of poorer rock quality, and kinematically possible failure conditions. Construction contract provisions for pioneering work, slope flattening and additional right-of-way, as well as allowance for stabilization measures such as rock bolts/dowels, shotcrete, scaling etc., need also to be considered during the design and contract development phases of the project. Proactive geotechnical participation throughout construction, particularly during pioneering and first lift excavation, is also critical to this project phase. These design and construction issues are augmented by experience on three recent projects:

- Major realignment of State Route 395 north of Spokane, Washington requiring up to 200 foot high rock cuts in a structurally controlled, weathered granite.

- Major widening of State Route 12 in Aberdeen, Washington requiring up to 250 foot high rock cuts in a massive, very weak sandstone.

- Proposed realignment, widening and bridge/retaining wall replacements on State Route 12 west of White Pass, Washington requiring rock excavations and rockfall control measures in bedded and intrusive volcanics for cuts up to 100 feet in height.

INTRODUCTION

Rock excavations in steep ground for transportation facilities require special geotechnical considerations during all phases of the project to minimize construction cost overruns and risks, and to provide safe finished slopes. Commonly, geotechnical input is only sought during the design phase of a project and when problems arise on construction. Recent experience has shown that geotechnical input provided during project scoping can avert protracted design efforts and later construction problems. Also, proactive geotechnical participation on construction can prevent or significantly limit impacts of unanticipated, adverse ground conditions. This paper discusses geotechnical concerns for rock excavations in steep ground during each major phase of a project. Several recent highway projects in

Washington State, currently in various project phases, illustrate the importance of these perhaps obvious geotechnical considerations.

PROJECT SCOPING

Few new highway alignments are being constructed in Washington State. The majority of projects entail widenings and minor geometric improvements and structure replacements. For new alignments, geotechnical input is typically sought by the design engineer early on in the project development phase. However, in a number of cases, the project scoping for the widenings and/or "minor" alignment revisions are developed with little geotechnical input. (It is critical that those providing geotechnical input be experienced in rock slope engineering in both design and construction.) In these instances, the geotechnical specialist's first involvement with the project is in the design phase when they receive alignment plans and profiles, and perhaps, a few alignment alternatives to evaluate. When projects are located in steep ground, an alignment shift or widening of just a few feet can result in significant geotechnical impacts. For this reason, it is imperative that geotechnical input is provided during this phase of a project.

It is not unusual during the project scoping phase that design goals are included that address perceived or non significant problems, or more commonly, fail to anticipate resultant impacts of these design goals. This is particularly true for rockfall and constructibility issues. In one recent project, a rockfall ditch was constructed for a several thousand foot long section of roadway. Utilizing the existing alignment, the ditch required widening into a steep hillside resulting in cuts 50 to 80 feet in height. Pioneering efforts to access the top of the cut were difficult, exceeding what both the State and the contractor had anticipated. The unfortunate point is that prior to construction there were minimal rockfall hazards at this site.

Constructibility issues and evaluation of risk should be thoroughly discussed during the project scoping phase. In steep ground, alignment shifts and widenings typically balance excavations on the uphill side against sliver fills, retaining walls, and bridges for the downhill side. Admittedly, structure costs are high. This is the common driver for moving alignments upslope. However, pioneering and excavation costs as well as additional costs for necessary rock slope stabilization can easily exceed structure costs for downslope widening. Variable subsurface conditions are common in steep ground. This variability is often difficult to define during the geotechnical investigation, and can result in serious construction and cost impacts later in the project. What would the impacts be if overburden thickness is 20 feet instead of 5 feet? In steep ground, this could result in very significant right-of-way impacts, increased excavation quantities, and the need for stabilization items such as wire mesh. How would the unanticipated presence of adverse rock structure effect the project? In a recently constructed project near Spokane, Washington, one adverse plane resulted in a very large slope failure requiring that a large portion of the slope in excess of 150 feet in height be flattened from a 0.5H:1V to a 1.5H:1V and the incorporation of a shot-in-place buttress at the toe of the slope.

DESIGN

The design philosophy for rock excavations utilized by Washington State Department of Transportation (WSDOT) is to construct cut slopes as steeply and as uniformly as possible while attempting to minimize cut slope heights, excavation quantities, and rockfall hazards. Rockfall ditches are provided, when feasible, and controlled blasting methods are required for all cut slopes steeper than 0.5H:1V. Midslope benches are not utilized except when required for difficult access, particular slope geometries, or rock mass conditions. Then, they are utilized only in the upper portions of the cut. This design philosophy for rock excavations in steep ground requires a very thorough assessment of ground conditions, which may or may not include geotechnical drilling. Kinematic evaluation of the geologic structure is always critical to this investigation.

Once the initial alignment has been developed, detailed geotechnical design review, investigation and recommendations are required. Typically, an early review of the cross sections with the cursory roadway template is done. In steep ground (particularly slopes with thick vegetative cover), photogrammetrically developed cross sections can contain significant errors. Ground truthing these cross sections is very important. Design catchpoints should be staked in the field prior to this initial geotechnical walk-through. This initial field review provides site specific information for development of the geotechnical investigation plan. At this time, evaluation of specific geotechnical issues should be considered. In addition to the geotechnical issues common to most excavations (i.e., material properties, ground water, etc.), the following design issues are of paramount importance to rock excavations in steep ground:

- variability in condition and thickness of overburden/highly weathered rock;
- adverse geologic structure that could result in small or large scale slope failures;
- potential impacts of large scale slope failures; and,
- pioneering efforts and constructibility issues.

It is likely that thicknesses and condition of the overburden and the upper weathered rind of bedrock will be highly variable. Effort spent delineating overburden thickness and condition, or potential ranges of variability, during the design phase will likely be rewarded with fewer expensive surprises during construction. On occasion, consideration for overburden has been neglected or minimized in the final contract pl!3ò The contract plans should be carefully reviewed to insure this important issue is addressed. In a recent widening project near Aberdeen, Washington, a 15 to 25 foot thick layer of highly weathered sandstone and overburden was anticipated based upon the geotechnical investigation. A compound slope was shown in the contract plans showing an upper 1.5H:1V slope for the weathered rock/overburden soils with a 0.5H:1V in the fresher sandstone. Unfortunately, the location of the hinge point was not located in the plans and the overburden quantities were never estimated and included in the planned quantities. And when unanticipated variability in the quality and thicknesses of highly weathered rock and overburden was encountered in the eastern portion of the cut, a redesign was required resulting in an additional 80,000 yds^3 of excavation and the purchase of additional right-of-way.

One of the first geologic assessments made for any rock excavation is determining if the rock mass contains repeating and/or continuous joint sets, which is commonly referred to as structural control. If it does, then a thorough kinematic evaluation is necessary to determine the pervasiveness and adversity of this structure to the overall stability of the cut. An assessment of shear strengths of the discontinuities will be required, which will likely necessitate performing shear strength tests. Back slopes are then designed to cut out as many of the potentially unstable joint sets as possible while still attempting to minimize excavation quantities.

Despite thorough geotechnical investigations, large scale failures/instabilities are an ever-present risk in rock excavations. Rock is rarely homogenous in its engineering properties. Parametric and sensitivity analyses need to be performed for critical design data and assumptions to better evaluate risks. The presence of one adverse, low strength discontinuity, easily not detected by drilling or field mapping, can result in a large failure. This situation occurred while constructing an approximately 100 foot high 0.5H:1V cut in a granite near Spokane. Slight variation in the orientation of a pervasive and continuous, favorably oriented joint set resulted in a very large slope failure. The only feasible remediation of the slope after the failure was to reconstruct the slope coincident with this joint set (approximately a 1.5H:1V slope) and place a large shot-in-place buttress section in the ditch. This remediation dramatically increased excavation quantities and the finished slope height, and it impacted the contractor's work schedule.

Constructibility issues that could impact the proposed design also should be investigated. Access and pioneering efforts can be extreme in steep ground, rendering some designs virtually unconstructible. For this reason, it is very helpful to have the proposed catchpoints staked in the field. The cross sections and roadway templates also should be reviewed to determine proposed working widths of cuts. A minimum bench width of 15 to 20 feet is generally necessary to allow for working area for multiple lift excavations. Sometimes a muck pile can be constructed to accommodate a decrease in width, but generally this is not feasible until the lower lifts of an excavation. A current design project located in mountainous terrain near White Pass, Washington, calls for replacement of three bridges and widening of the narrow roadway section. Initially, the design planned for moving the roadway upslope to shorten bridge spans and avoid the construction of additional retaining walls on the downhill side. This initial plan included several thousand feet of thin sliver cuts (less than 15 feet wide), many in excess of 100 feet in height. Equipment access to the proposed catches were, in many locations, impossible. Through close interaction between the design office and the geotechnical and structural designers, the final design has evolved to dramatically reduce the size and quantity of rock excavations, increase the bridge spans slightly, add several new retaining walls, and provide deviations for required width in several difficult locations.

Topography of the existing ground, sometimes a neglected source of design data, can provide very useful information in the development of the final slope design. Genesis of drainage and other topographic features are commonly a function of the site geology and, ultimately, the engineering properties of the subsurface materials. In several cases of recent excavations in Spokane and Aberdeen, Washington, poor ground conditions (i.e., thick overburden deposits, poorer quality rock, etc.) were encountered in swale areas that were not encountered in areas of more uniform topography. The excavation of an extensive zone of moderately to highly weathered, massive, very weak sandstone in Aberdeen, revealed that the maximum stable slope was identical to the existing natural slope (approximately a 1H:1V slope).

CONTRACT DEVELOPMENT

Preparation of the contract plans, specifications and estimates (PS&E) for a project should be as inclusive and accurate of the actual work that will be performed as possible. For rock excavations in steep ground, anticipating and providing for likely difficult ground conditions lessens construction costs and impacts.

Construction easements and long term maintenance easements, outside of the planned right-of-way, may be required. The need for and extent of these easements should be based upon assessment of the pioneering efforts that will be required and the condition of the ground in the first lift of the excavation. Proceeding into construction without these easements is risky, and if needed later may result in a significant delay during construction.

For most all multiple lift excavations, WSDOT includes an advisory specification in the contract that informs the contractor that WSDOT will inspect the initial clearing and grubbing and each lift of the excavation as the cut is brought down to grade. WSDOT will determine at that time if additional stabilization work is required. The contractor must then perform this work prior to the excavation of the next lift. This specification forms a contractual working relationship with the geotechnical specialist, the construction office and the contractor. Stabilization that is delayed until after completion of the excavation exposes the contractor and the inspection staff to unnecessary risks, is typically much more expensive to perform, and frequently is minimized or not performed at all.

For rock excavations that may encounter adverse structure, loose rock debris outside the planned catch limits, localized highly erodible zones, etc., WSDOT includes bid items for anticipated stabilization work in the contract. These items may include scaling, rock bolting and doweling, shotcrete, wire/cable mesh, rockfall fences, etc. Even though some quantities may be nothing more than a "best guess", having anticipated work items in the contract alerts the contractor that these conditions should be expected. The bid items also serve as a basis of cost negotiation to perform the work if the work is needed and if quantity overruns/underruns occur.

If construction is to be performed under traffic, consideration needs to be given to setting up traffic control or detours, establishing work windows, and providing temporary debris barriers. Although the cost of temporary barriers may initially seem high, these costs should be weighed against shortened work windows, increased work duration, and additional traffic control costs. The use of temporary debris barriers to maintain traffic flow during construction, will require a very careful assessment of rockfall behavior both prior to and during construction as the slope configuration changes.

CONSTRUCTION

It is imprudent to rely on the construction inspection staff or the contractor to make determinations regarding rock slope stability. These determinations need to be made by a geotechnical specialist who is experienced in rock slope design, construction, and remediation. The advisory specification described above provides a contractual framework for the relationship between the geotechnical specialist, the construction inspection staff, and the contractor. However, it is primarily the responsibility of the inspection staff to actuate this relationship. It is imperative that there be a mutual understanding and appreciation of each parties concerns and experience base for these type of projects. Partnering appears to be a very valuable process in forging this relationship from the beginning of construction.

Once construction begins, the geotechnical specialist needs to be available on relatively short notice to review the progress of the work. Scheduling of geotechnical reviews and performing necessary stabilization are integral to the main excavation work. And it is assured that contractors will be focusing on where they can make their money, moving material. Therefore, it is important that the geotechnical specialist be capable and prepared to assess conditions and make recommendations expeditiously. Even minor delays to a contractor's scheduled work plan can result in extremely expensive impacts for this type of construction.

Regular construction reviews by the geotechnical specialist also provides an opportunity to review design assumptions. Initial construction reviews should be made after clearing and grubbing, during pioneering, and prior to excavation of the first lift. At this time, the final design catches can be evaluated and necessary changes (i.e., setbacks, overburden stabilization, etc.) made prior to beginning construction of the design slope. The recent excavation of 200 foot high slope near Spokane necessitated the frequent placement of rock bolts to reinforce localized unstable blocks. The rock bolting subcontractor could reach approximately 15 to 20 feet with an extendible boom air track drill. Since the earthwork contractor was excavating 40 foot lifts, a geotechnical field review was planned every half lift of excavation. Although this did require almost constant coordination between all parties, the arrangement worked well and minimized delays for the contractors.

SUMMARY

The high construction costs, potential impact of road closures due to slope failures, and the long term safety of finished rock slopes in steep ground, demand that the geotechnical specialist be involved from the beginning stages through construction of the project. The critical role for the geotechnical specialist during the project scoping phase is to provide input and evaluate risks for design alternatives, clarify project goals, and evaluate constructibility issues. During the design phase, cross sections and proposed roadway templates need to be reviewed in the field. Particular attention should be paid to variable overburden depths, adverse geologic structure, the potential impacts of slope failures, and constructibility. Provisions need to be made in the contract to address these conditions, which may include enlarged construction easements and inclusion of stabilization measures such as rock bolts and dowels, shotcrete, scaling, etc. The geotechnical specialist needs to play a very proactive and visible role during construction. A strong, communicative relationship between the contractor, inspection staff, and the geotechnical specialist is very important to insure prompt remediation of any problems while minimizing impacts to all parties.

APPLICATION OF THE ROCKFALL HAZARD RATING SYSTEM TO ROCK CUTS IN MOUNTAINOUS TERRAIN

by

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ABSTRACT

The Rockfall Hazard Rating System (RHRS), developed by the Oregon Department of Transportation, a method to rank slopes according to their rockfall hazard, was utilized to evaluate highway slopes adjacent to US 50 and State Route 28, on the east side of Lake Tahoe, Nevada. Rock slopes in the area have many documented cases of accidents resulting from rockfall. Two major rock units, granodiorite and latite, in various stages of alteration and weathering, comprise the lithologies on the east and north sides of Lake Tahoe. Given the Lake elevation at over 6000 feet, in the Sierra Nevada, climatic effects are significant with snow fall averaging 100 to 200 inches per year at lake level, diurnal temperature changes approaching 40 degrees fahrenheit, and seasonal thunderstorms. Over 115 individual rock slopes were appraised, with particular attention being addressed to the unique conditions which slopes experience in mountainous areas. High precipitation rates and cold winters result in both abundant water in and on the slopes with severe frost wedging during the winter and in the early spring months.

The results of the study were two fold. Firstly, 35 slopes were highlighted which could produce hazardous rockfall, and mitigations were developed to minimize and reduce the hazard. Secondly, modifications were made to the RHRS system to increase, a) the effectiveness and objectivity of the system, and b) assess climatic and slope aspect (i.e. slope azimuth) effects on rockfall. Slope height, ditch effectiveness, and average vehicle risk categories were altered . A new category which includes climatic and slope aspect affects was added to effect the overall rating. The new category takes account of snow depth in ditches, which reduces ditch effectiveness, and slope aspect was found to increase or decrease rockfall.

ROCK RETAINING WIRE ROPE NET SYSTEM INSTALLATION

Orizaba, Mexico

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ABSTRACT

The wire rope net barrier at the Cementos Apasco site is the first rockfall retaining system installed in Mexico (Figure - 1). Emphasis is placed on the surveying and design criteria, as well as the construction of the defensive barrier. Additionally, the paper focuses on the difficulty of the installation and the transport of material to the construction site which had to be done manually. A brief discussion is given concerning the design and the attention given by Cementos Apasco with regard to aesthetics of these rock retaining fences and the importance of realizing the general safety these rockfall fences provide for the general public as well as goods.

Figure - 1

INTRODUCTION

In order to ensure future availability of raw materials (limestone) to the Orizaba plant of Cementos Apasco in the state of Vera Cruz, Mexico, this company is currently executing a large project to open a new quarry in the mountains that are in close proximity to the Company's cement plant as well as to the nearby cities of Ixtacoquitlan and Orizaba, Mexico (Figure - 2).

In 1993, during the initial execution stage of the project, several incidents of medium size rocks rolling downhill caused fears by those whose homes were located about 800 meters (2625 feet) from the project site at the base of the mountain on the down side of

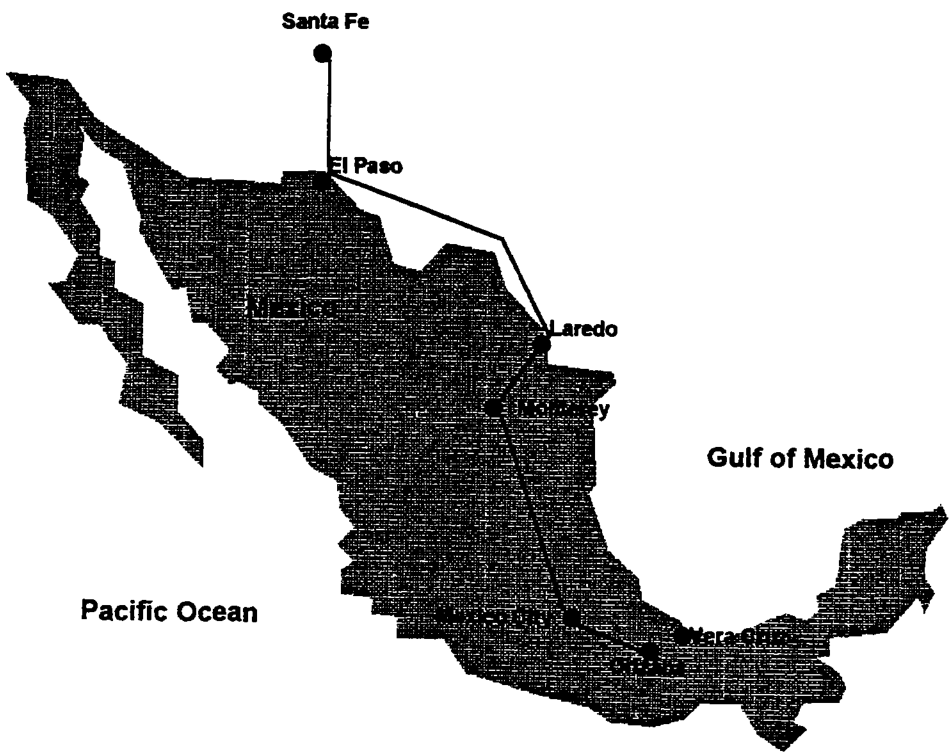

Figure - 2

Cuautlapan. It should be noted that the danger of rockfall was not created by natural occurrences, but mainly due to the activities relating to obtaining limestone from the mountain to be used in the cement making process. At about the same time, another incident occurred at the company's existing quarry located on the neighboring mountain. A boulder weighing approximately 5.18 tons came rolling down the mountain slope and scraped a bus driving through the public road located downhill of the operating site. Fortunately, no one was hurt but the incident created significant outcry among the public and from local politicians.

Because of safety as well as political and community relation reasons, the company immediately proceeded to investigate the rockfall protection systems suitable for their application. After evaluating all the possible vendors and carefully analyzing the locations with regard to where the rock retaining fences needed to be placed, (Figure - 3), Brugg Cable Products, Inc. in Santa Fe, New Mexico was contacted in order to prepare a conceptual design study and submit an economic proposal for Cementos Apasco's solution. The proposed fences which were to be located in critical areas at the new quarry have been included as part of the environmental impact statement prepared for this project which was submitted to the proper authorities by Cementos Apasco.

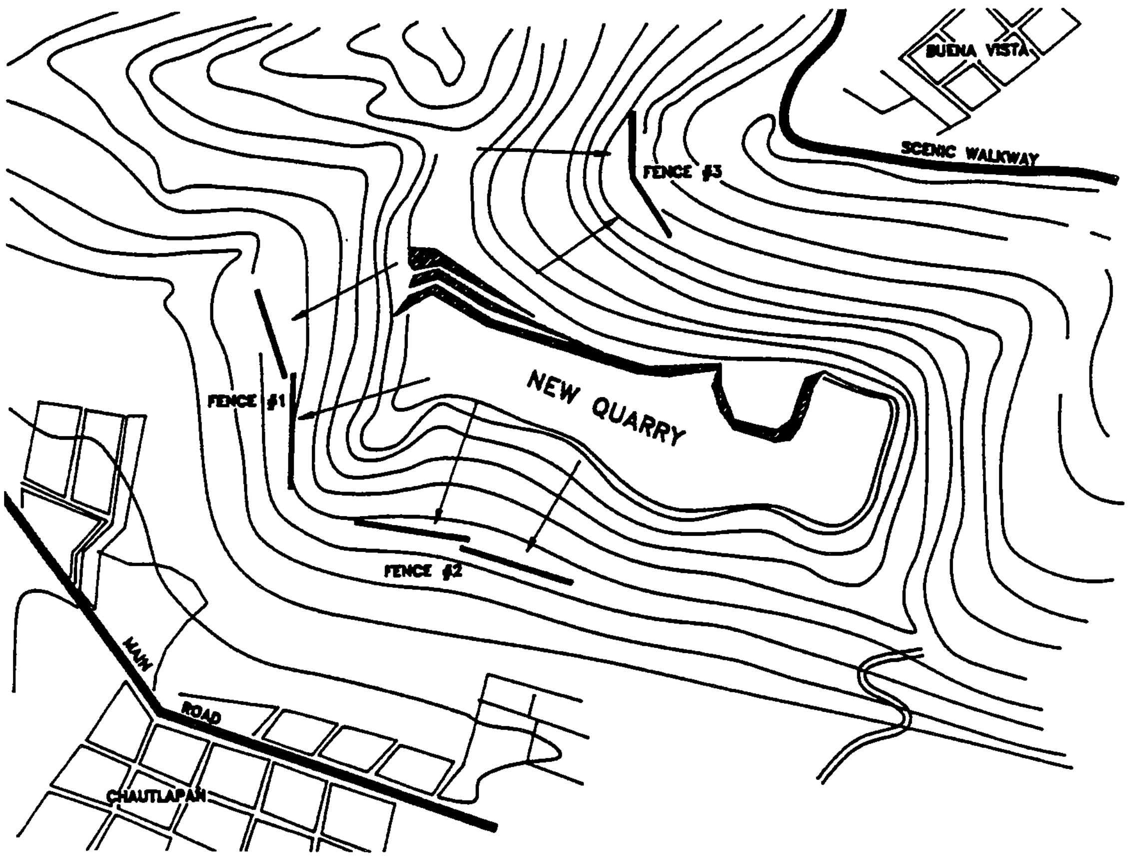

Figure - 3

ROCKFALL SYSTEM SPECIFICATION AND DESIGN

Phase I of the project consisted of three rockfall fence systems of which two fences (#1 & #2) were to be permanently installed and one of the fence systems was to be moveable. This meant that once the area ceased to be a rockfall hazard, the system would then be moved to the next danger zone and reused. Phase II of the project required that an additional four sections of moveable rock retaining systems would be installed at a later date and therefore would not be discussed in this particular paper.

In order to calculate and establish the design criteria for these rockfall systems, the Colorado Department of Highway's "Colorado Rockfall Simulation Program" was used. This program was developed to provide a statistical analysis of probable rockfall behavior at given sites. Considering the dense jungle type vegetation and in view of the inaccessibility of the sites, much thought was given regarding the overall design of the system. The barrier must be constructed from standard, lightweight components for rapid and easy installation on site. Sufficient safety was to be included when determining the design, mainly to keep maintenance to a minimum (Figure - 4).

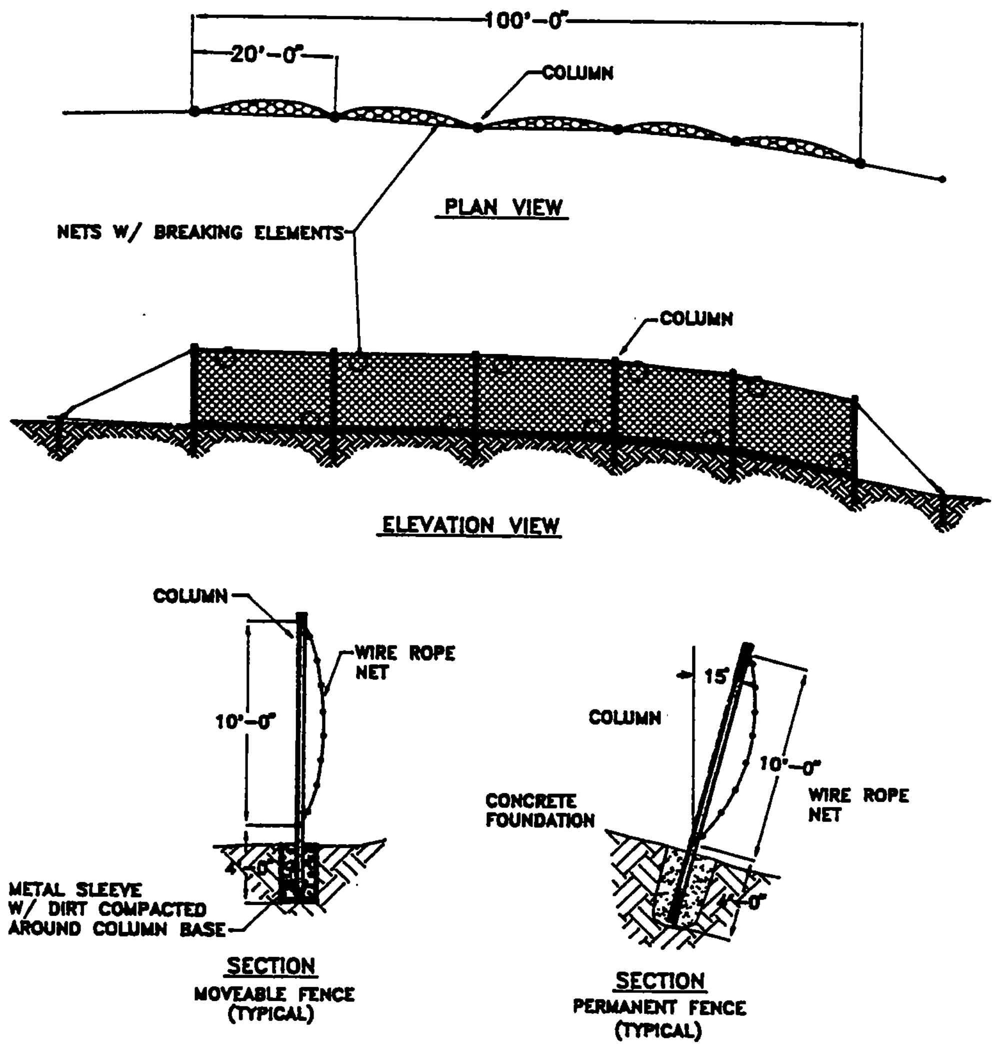

Figure - 4

The basic rockfall protection barrier design data is as follows:

- Angle of Slopes	48 ° to 60 °
- Maximum Weight of Rocks	5.18 Tons
- Maximum Impact Energy the System Can Absorb	129 Ft- Tons

- Average Jumping Height of the Falling Rock	3 - 4 Feet
- Average Velocity in Relation to the Maximum Height and Acceleration Due to Gravity	32.85 Ft. / Second
- Ground Surface	Mostly soft moraine with some rock formations.
- Vegetation in Surrounding Areas	Large tropical trees including banana and coffee trees, scrub oak and other bushes.

Numerous field trips were required prior to preparation of the conceptual study and commercial proposal. In order to reduce the cost of materials and the subsequent freight and duty, as well as other related expenses, scrap railroad steel rails were used for the system support columns (Figure - 5), which were readily available for no cost at

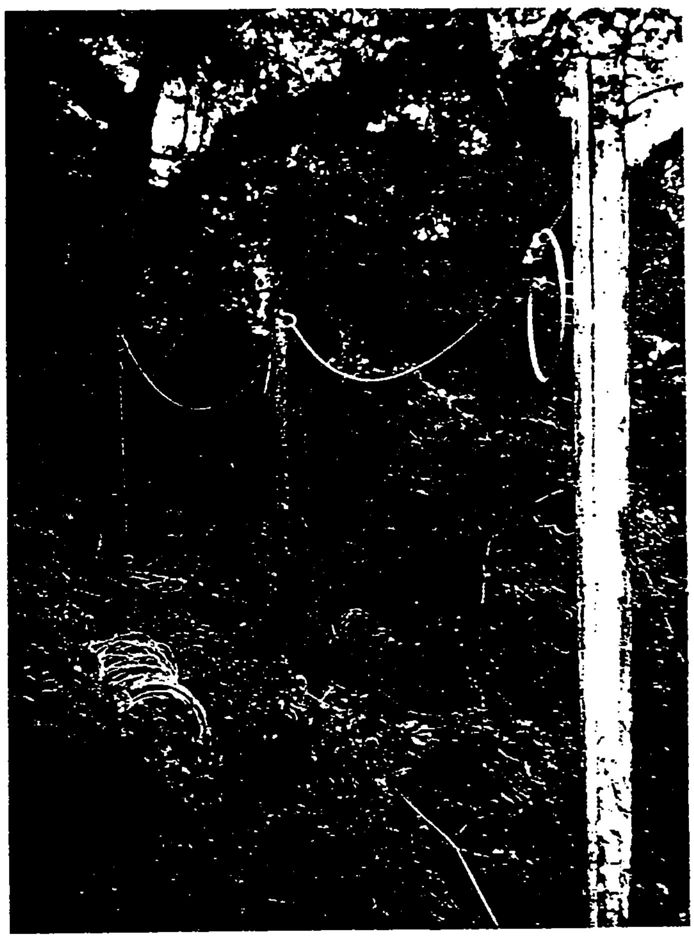

Figure - 5

the Cementos Apasco plant. All cutting , drilling, ect. for these columns was executed in accordance with the plan and specifications by Cementos Apasco personnel.

The system chosen for these danger zones, in accordance with our test results, was to be capable of absorbing an energy of 129 ft-tons, thereby allowing for a design which provided more than adequate strength.

TRANSPORTATION AND INSTALLATION OF THE ROCKFALL SYSTEM

In order to finalize the detailed design of the rockfall retaining system, the surveying of the project sites and consequently, the staking out of the system began immediately. It should be noted that in order to get this work done, the area where the proposed fences were to be located had to be cleared first by hand with machetes used by the local workers, which in itself was quite some task (Figure - 6).

Figure - 6

The layout of the system was determined by surveying the area, using the COBOL Land Surveying program which works in conjunction with the PC Computer Auto CAD program.

After clearing of the fence line was accomplished, the column location points were staked out and then located by use of a theodolite instrument. This instrument locates the X, Y and Z coordinates of the points and stores them in a hand held computer. After returning to the company's offices, data from the field computer was loaded directly into a desk PC computer were it was manipulated by the Auto CAD program. From there it was a simple matter of displaying the coordinates in three dimensional space so that the exact locations of the columns could be obtained as well as the shapes of the required nets between the columns. All design documents were then prepared from the data collected in the field. The total length of these fences was 1116 meters (3660 feet). Many angular nets had to be fabricated so they would fit the contour of the mountain. The heights of all the nets is 3 meters (approx. 10 feet).

The required materials with the exception of the steel columns were manufactured in the vendors plant in Santa Fe, New Mexico and shipped via truck to the Cementos Apasco plant. As mentioned previously, since the location of the fences is in the mountainous areas and not easily accessed (Figure - 7), no heavy mechanical

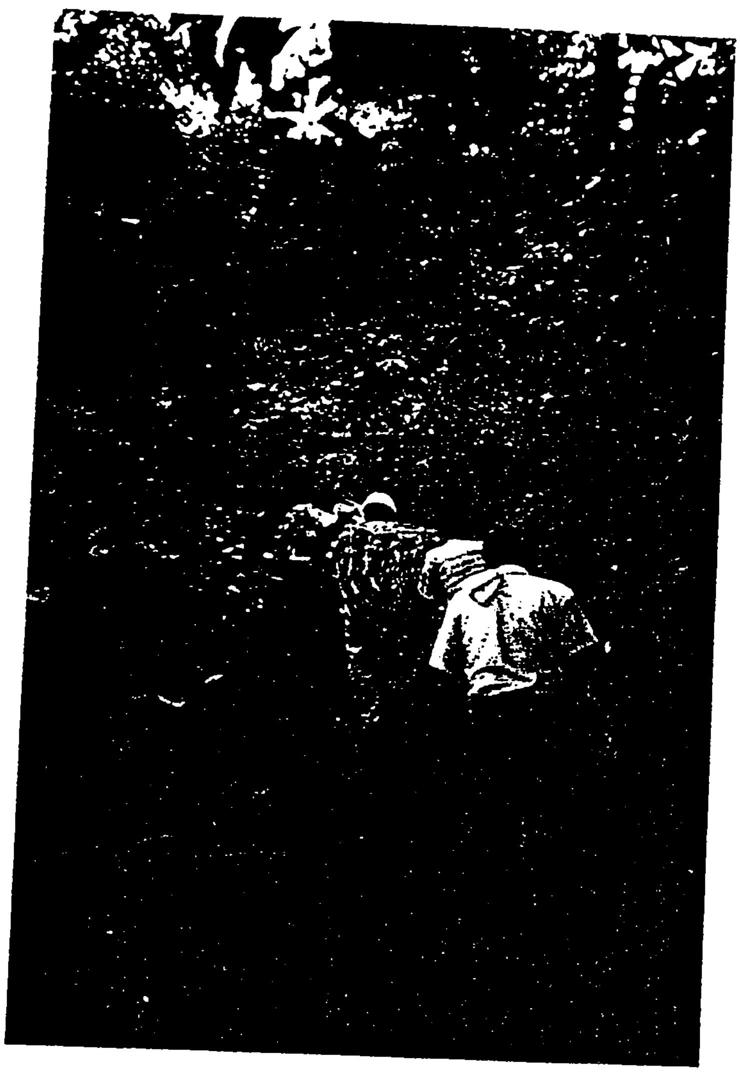

Figure -7

machinery could reach the installation site. Excavation of the foundations was done manually using only gasoline powered percussion drills for the rock excavation encountered. All materials required for the retaining system's installation such as cement and gravel, steel columns, nets and wire ropes, ect. had to be manually transported up the mountainside.

The concrete mixing required for the steel column foundations for the permanently located retaining fences was done entirely by hand and so was the installation of the net system and by using the simplest of tools (Figure - 8). No concrete foundations were needed for the moveable retaining fence. The steel columns were simply set into sleeves, resting about four feet below the ground. Considering the difficult conditions, the installation progressed smoothly and on time.

Figure - 8

During the five months it took to finish the installation, an average of 14 workers were required for the installation and additionally, approximately 12 people were needed on and off to manually transport the required materials and tools to the project site.

CONCLUSION

We would like to give special consideration to Cementos Apasco for their effort in having specified a system which not only is quite effective but also aesthetically and environmentally pleasing. Of equal importance is the fact that they had the foresight to realize the importance of the general safety that these systems provide for the general public as well as goods. Aside from good planning, we also received excellent data pertaining to the danger zones from peoole familiar with the area. (Figure - 9).

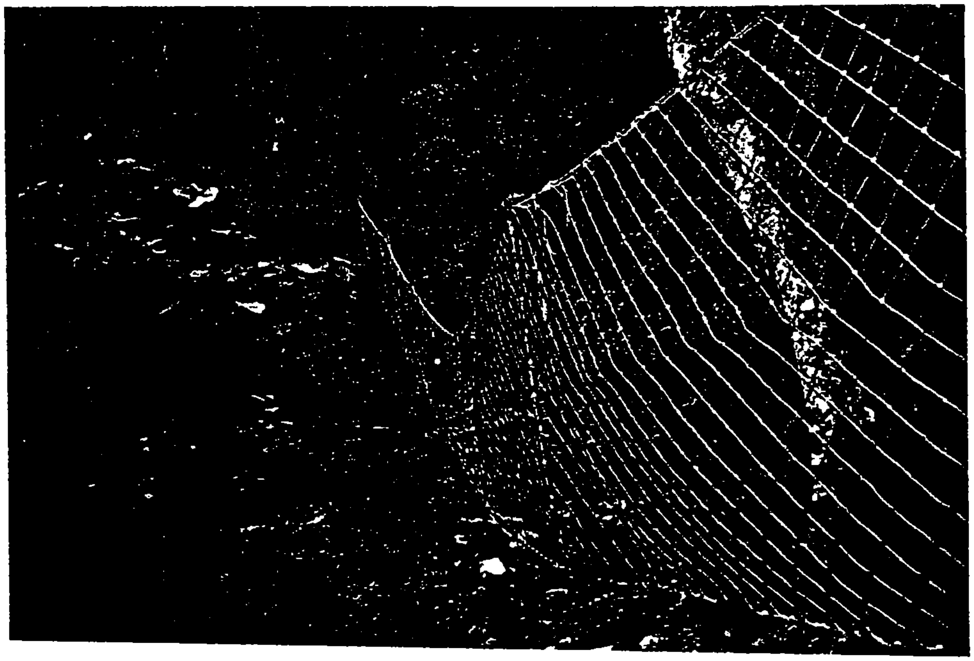

Figure - 9

REFERENCES

- California Department of Transportation
  Duane D. Smith and John Duffy
  "Field Test and Evaluations of Rockfall Restraining Nets"
  (Report No. CA/TL-90/05)

- Colorado Department of Transportation
  Bob Barrett and Tim Bower
  "Colorado Rockfall Simulation Program"
  (CDOT-DTD-EDS/CSM-89-2B)

DESIGN, CONSTRUCTION, AND PERFORMANCE OF HORIZONTAL DRAINS AT BONNEVILLE NAVIGATION LOCK, OREGON.

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ABSTRACT

Two drainage shafts with arrays of radial horizontal drains were installed in 1991 at the toe of the Tooth Rock Landslide in the Columbia Gorge. This drainage system is part of the new Navigation Lock at Bonneville Lock and Dam. The upstream approach channel and Guard Wall for the lock crosses the toe of the Tooth Rock Landslide. Horizontal drains were constructed to lower ground water level and improve the existing stability of the toe of the landslide mass above the approach channel. One hundred and twenty three drains were drilled. Lengths of production drains varied from 200 to more than 600 feet for a total of approximately 56,000 lineal feet of drains.

The Tooth Rock Landslide is a large ancient landslide. This landslide has been investigated by a series of more than 28 borings and five horizontal test drains by the Corps of Engineers. The ancient landslide mass is about 3,000 feet long and 4,000 feet wide. Subsurface investigations suggest that the landslide is a deep (maximum depth of failure plane approximately 1000 feet) rotational slump failure. The age of landsliding is believed to be pre-Missoula Floods (greater than 18,000 years). However, a smaller landslide occurred at the toe in 1937 during relocation of the Union Pacific Railroad line. The toe of the landslide is characterized by a chaotic mixture of large landslide blocks separated by variable mixtures of clay, silt, and sand matrix derived from the granulation and decomposition of the sedimentary units. Blocks larger than 300 feet across have been delineated both on the surface and in the subsurface.

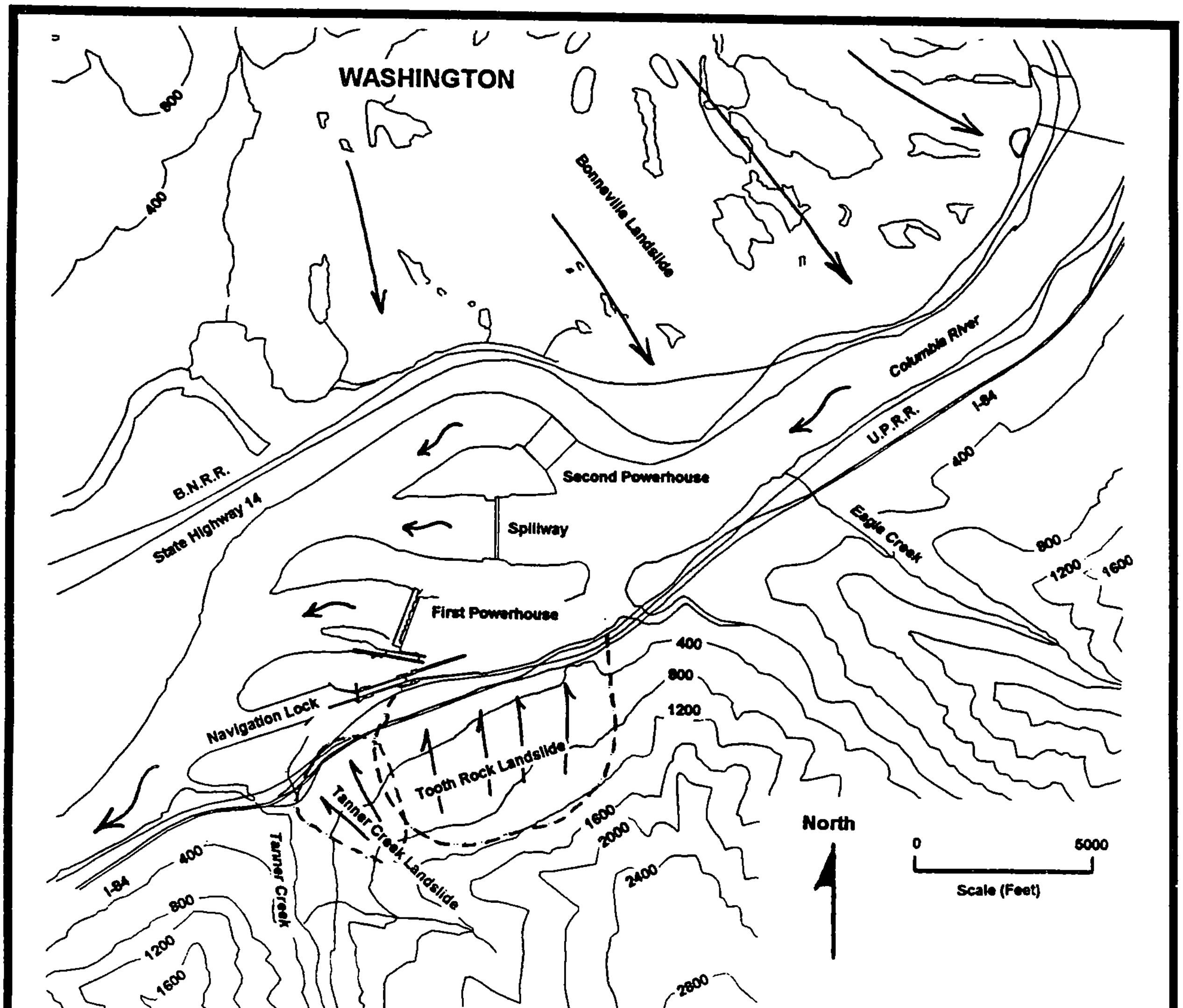
The design of the drainage system was based on a geotechnical investigation program that included approximately 31 borings south of I-84, five test horizontal drains, and one aquifer test. In addition, more than The longest test drain was 740 feet. The as-built drainage system consists of two 14-foot diameter drainage shafts. A total of one hundred twenty three drains were drilled at multiple levels with two arrays in each shaft. The drainage shafts are connected by a tunnel that permits drainage by gravity.

The drains have performed well. They have lowered the ground water level 20 to 40 feet behind the Guard Wall in the toe of the landslide. Summer flows range between 50 and 80 gallons per minute. Peak winter flow up to 300 gallons per minute has been measured. Integration of the drainage flow shows that about 80 percent of the winter precipitation was drained during a four month period.

INTRODUCTION

The Bonneville Lock and Dam is on the Columbia River about 42 miles east of Portland. The Bonneville Project consists of a spillway dam, two powerhouses, and a navigation lock (Figure 1). The original powerhouse, southern portion of the spillway, and navigation lock are located in Oregon. The northern half of the spillway and the second powerhouse are in Washington. The Bonneville Project was constructed primarily for hydroelectric power production and improvement of navigation. The original project was completed in 1937. Since then, it has been enlarged with completion of the second powerhouse in 1982 and the replacement navigation lock in 1993.

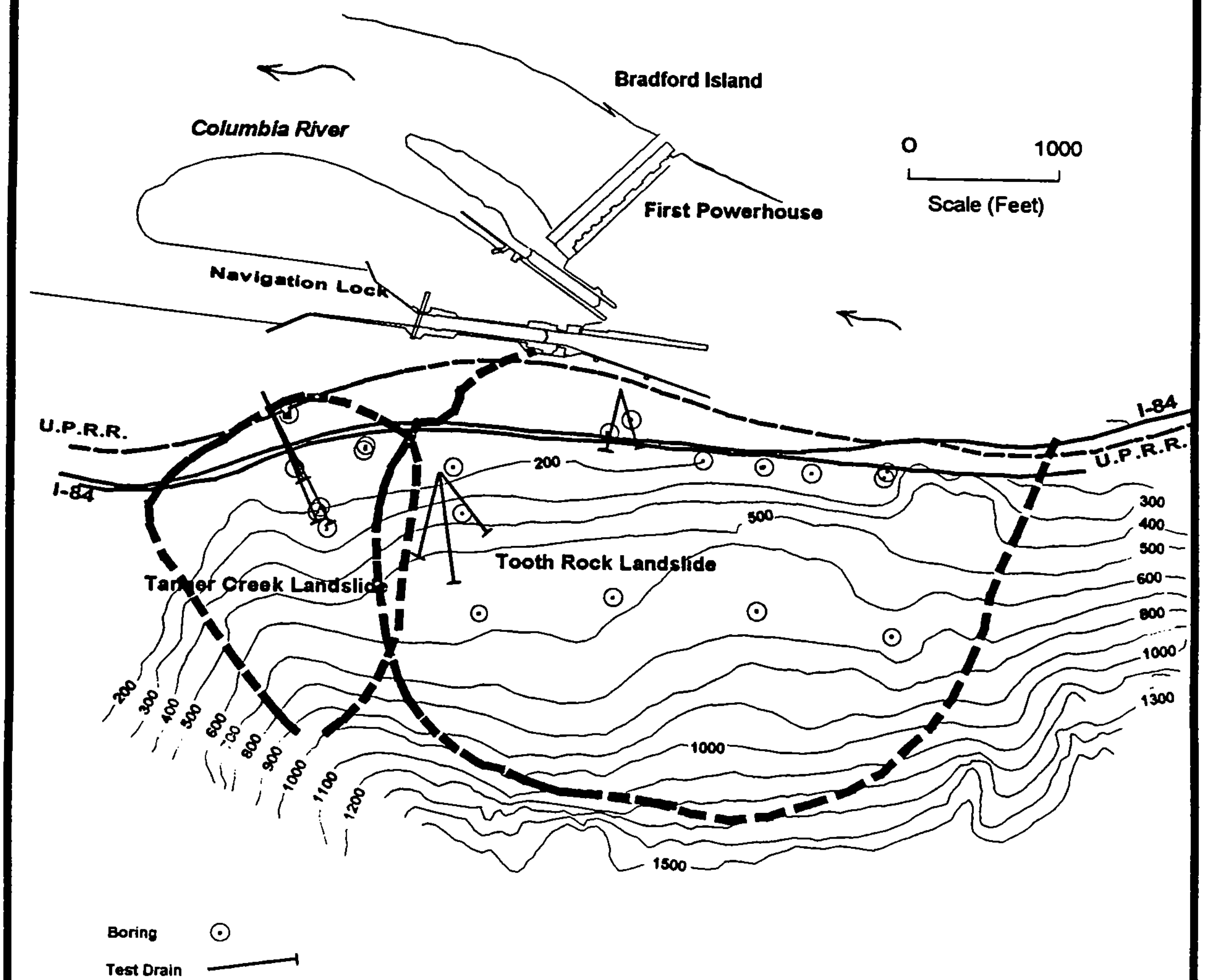
The Bonneville Project is located in the Columbia Gorge. The Columbia Gorge is a 50-mile long canyon cut through the Cascade Range by the Columbia River. The Columbia Gorge provides a major transportation corridor through the Cascade Range for interstate highway, rail, navigation, and electric power transmission.



LOCATION MAP
FIGURE 1

Purpose

This paper focuses on the explorations, design considerations, construction, monitoring, and performance of horizontal drains installed in the toe of the Tooth Rock Landslide on the Oregon side as part of the new navigation lock project (Figure 2). These drains were installed to lower the hydrostatic pressures behind the Guard Wall and improving the stability of the toe of the landslide. The Guard Wall is a major 70-foot high reinforced, tied-back concrete wall constructed in the toe of the landslide. The landslide has been instrumented and monitored with slope inclinometers and piezometers. In addition, the Tooth Rock drainage system has been instrumented and monitored. This instrumentation provides an opportunity to measure the performance of the drainage system. It also provides information regarding seasonal recharge characteristics of the Tooth Rock slide.



EXPLORATIONS AND TEST DRAINS
FIGURE 2

Climate

The Bonneville Project area has a temperate climate that is characterized by cool, moist winters and warm dry summers. Daily temperature and precipitation data are recorded on the Oregon side of the project site. The table below shows the monthly average temperature and precipitation since 1937. Also, the maximum and minimum monthly precipitation and the greatest one day precipitation records are shown. The long term average precipitation measured at the site is 76.52 inches annually. Nearly 70 percent of this occurs as rain during the winter months of November through March.

SUMMARY OF CLIMATIC CONDITIONS AT BONNEVILLE													
	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.	Average Annual
Temperature													
Average	36.8	41.3	44.5	50.0	56.2	61.8	67.2	66.9	62.9	54.5	45.0	39.7	52.3
Precipitation													
Average	12.59	8.56	8.29	5.29	3.69	2.44	0.75	1.74	2.94	6.35	10.57	13.31	76.52
Maximum Monthly	30.74	21.72	15.68	10.07	9.51	6.15	2.33	3.94	6.55	17.37	26.58	23.52	
Minimum Monthly	2.57	2.49	2.13	1.03	0.69	0.14	0.00	0.00	0.00	0.78	2.45	3.93	
Maximum Daily	4.34	4.11	4.00	4.85	2.33	2.74	1.60	2.08	2.52	6.40	4.40	5.05	

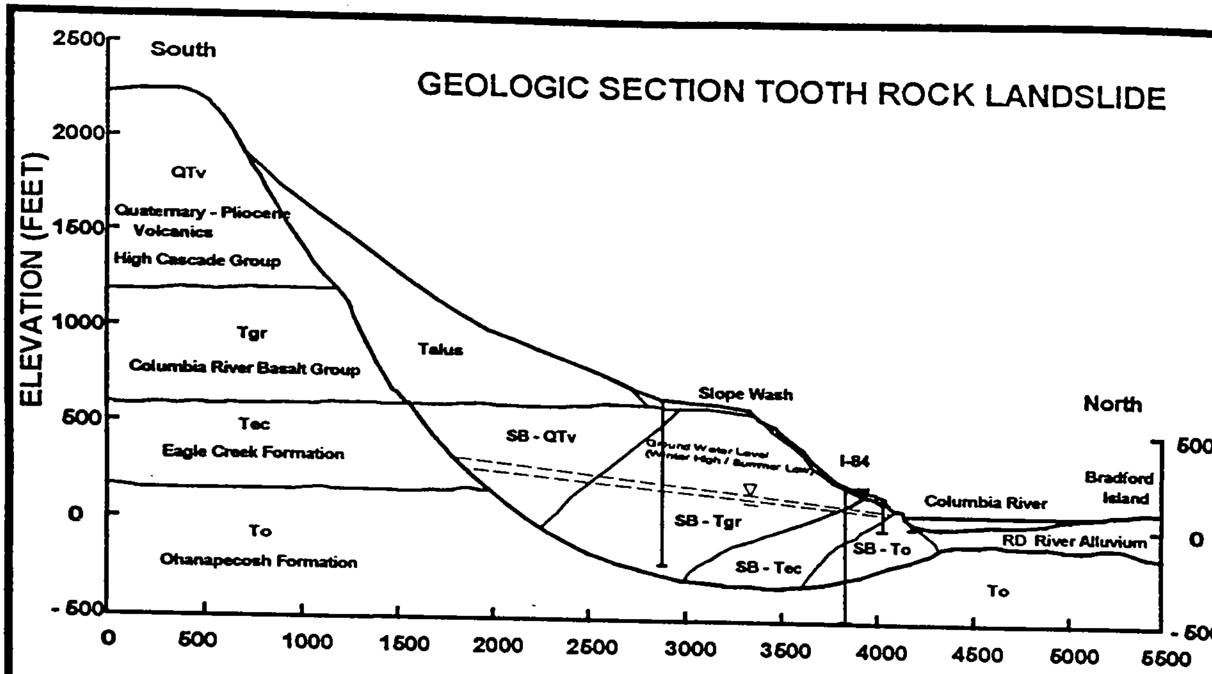
GEOLOGY

Bonneville project is situated along the Columbia River where the river crosses the Cascade Range. Over 3,000 feet of rock strata are exposed in the walls of the gorge (Figure 3). This sequence of strata records 40 million years of volcanism, sedimentation, and deformation. The oldest rock underlying the valley floor are lithologically similar (or equivalent) to the Ohanapecosh Formation (Wise, 1970; Hammond, 1980). The Ohanapecosh Formation in the Columbia Gorge area consists of tuffs, mudflows, debris flows, and related volcaniclastic sediments (Beaulieu, 1977). At Bonneville, it consists of a weakly indurated sedimentary sequence of siltstone and claystone with lesser amounts of sandstone and conglomerate (USCOE, 1984).

Eagle Creek Formation unconformably overlies the Ohanapecosh Formation (Wise, 1970). The unit is composed of a poorly indurated sequence of volcanic conglomerates, sandstones, and tuffs probably of Oligocene to early Miocene age. At Bonneville, the deposits consist of chaotic mixtures of cobble and boulder-size volcanic rocks in a fine grained matrix (USCOE, 1984).

Flows of the Columbia River Basalt Group unconformably overlie the Eagle Creek Formation. The Columbia River Basalt is mid to late Miocene in age. This unit forms the prominent cliffs of the gorge. The thickness of the basalt varies. At Bonneville, it is about 600 feet thick with the base of the unit about elevation 600 feet (USCOE, 1984). Consequently, this unit occurs on the sides of the canyon walls at Bonneville and at lower elevations in landslides.

Overlying the Columbia River Basalt Group are Pliocene volcanics of the High Cascade Volcanic Group. The Pliocene volcanics occur as scattered dikes and intrusives, and also forms the plateau capping the High Cascades. Younger volcanic accumulations form the present day topographic peaks. In the gorge, these volcanics consist mostly of basaltic lava flows and dikes. One major dike/sill intruding the Ohanapecosh/Eagle Creek Formations is found on the Oregon



EXPLANATION

Unconsolidated Materials - Soil Units are Delineated by light lines:

RD - Recent Columbia River alluvium includes stratified sand, gravel, cobble and boulder deposits with occasional silt and clay layers. Coarse fraction typically rounded consisting of Columbia River Basalt and lesser amounts of olivine basalt. Also contains metamorphic and granitic "exotics" and white mica derived from the Idaho batholith area. RD unit is generally present below El. 100 feet. However, a silty gravelly sand deposit with exotics is present on the El. 400 feet bench west of Tanner Creek. This deposit may be related the the Missoula Floods.

Talus - Angular rock fragments of predominately Columbia River Basalt Group and Quaternary - Pliocene Volcanic basalt forms deposits of GP to GM at the base of steep Tgr/QTv cliffs. Talus deposits cover the Tec-Tgr contract throughout most of the area.

Slope Wash and Talus - Brown to red-brown silty sand to sandy silt deposits. Unstratified, forms a thin cover over much of the area below the talus deposits. May contain varying amounts of angular rock fragments. This transported soil unit is probably derived from the chemical weathering of the olivine basalt lava flow.

Bedrock Units - Heavy lines indicate approximate and inferred contacts between major rock groups; light lines delineate areas of in-place outcrops for each rock group.

QTv - Plio-Pleistocene Volcanics of "High Cascade" type. consists of olivine basalt lava flows and flow breccias with local soil
SB-QTv interbeds between flows. Flows are horizontal. SB-QTv identifies intact landslide block(s) rotated greater than 30 degrees.

Tgr - Grande Ronde Formation of the Columbia River Basalt Group. Consists of basalt lava flows and flow breccias with local
SB-Tgr soil interbeds between flows. Flows are near subhorizontal. SB-Tgr iidentifies intact landslide block(s) rotated greater than 30 degrees.

Tec - Eagle Creek Formation consists of volcanic conglomerates and volcanic sandstones poorly bedded local siltstone and
SB-Tec sandstone interbeds. SB-Tec identifies intact landslide block(s) rotated greater than 30 degrees.

To - Ohanapecosh Formation (local informal name Weigle). Consists of massive mudstones with interbedded sandstones and
SB-To siltstones with local conglomerates. Materials are generally well bedded. Matrix is altered to clay minerals. Failure planes for major slides are seated in the unit. Maximum in-place bedding dip 30o. SB-To identifies landslide block(s) rotated greater than 30 degrees.

GEOLOGIC SECTION
FIGURE 3

side of the project (USCOE, 1984). The position of this dike/sill is a controlling factor in the location of the younger landslides. The youngest recognized geology/engineering units in the gorge are the Quaternary alluvial deposits along the river and a series of large landslides. The alluvial deposits consist of a heterogeneous, stratified sequence of alluvial sediments and reworked landslide debris that formed during the Pleistocence glacial period and the backfilling of the incised Columbia River channel following sea-level rise at the end of the Pleistocene. Major landsliding has occurred in the Columbia Gorge during and since the Pleistocene. The largest individual slide is the Bonneville Landslide which covers 14 square miles on the Washington side. The latest movement on the Bonneville slide has been dated about 700 years ago (Sager, 1993). The Oregon side of the gorge is characterized by a series of smaller coalescing landslides mostly less than 1 square mile in size. Two landslides located at the project are shown on Figure 2. These slides are the Tooth Rock and Tanner Creek Landslides and are the focus of this paper.

Geologic structure of the area is dominated by the broad up arching of the Cascade Range. Approximately 2,800 feet of structural relief exists on the top of the Columbia River Basalt Group between the axis of the Cascade arch and the Portland basin. The up arching has a gentle plunge to the south. At Bonneville, near the axis, the dominant attitude of the units is a gentle dip to the south.

TOOTH ROCK LANDSLIDE

The Tooth Rock Landslide is a complex slide with the main slide interpreted as a deep seated rotation slump and smaller blocks and slides at the toes. A geologic section through the landslide is shown on Figure 3. The age of the landslide predates the latest Pleistocene floods and is estimated to be 20,000 years. The main landslide is stable. Explorations suggests that the thickness of slide debris may exceed 1,000 feet. The toe area of the slide is characterized by complex discrete slide blocks in a soil matrix. The most recent movement occurred as a smaller slide at the toe during the construction of the original project and relocation of the railroad in 1937.

Explorations and Test Drains

The landslides were extensively explored with borings, test horizontal drains and one aquifer test (Figure 2). Approximately 31 borings were completed for the investigation of the toe of the landslide and for various proposed structural elements in the upstream approach channel. Not all of the borings are shown on Figure 2. In addition five test drains were constructed and monitored for a period of one year to determine the feasibility and constructibility of drains. Three drains were located in the main mass of the Tooth Rock Landslide, and two in the Railroad slide at the toe of the Tooth Rock Landslide. During this period, three other test drains were drilled in the adjoining Tanner Creek Landslide. The longest drain was 740 feet in Tooth Rock Landslide and nearly 1000 feet in Tanner Creek Landslide.

A five day aquifer test was conducted in the reworked slide deposit (RSD) in the upstream drainage and wall area. Twenty four piezometers were monitored. The slide debris had a transmissivity of 875 gallons per day per foot (hydraulic conductivity of 0.03 feet per minute) and a storage coefficient of 4×10^{-4}(dimensionless) (USCOE, 1984). The radius of influence for the well was calculated to be 2,600 feet based on interpretation of the aquifer test. However, the actual observed radius was only 850 feet. An additional 14 exploration borings and seven dewatering wells were completed during construction of the walls for the upstream approach channel. The dewatering wells were only capable of producing low flow rates from the landslide mass and reworked slide debris. Collective maximum sustained yield of all the construction dewatering wells was less than 100 gallons per minute during the winter.

Design Considerations

The drainage system was designed to lower and maintain ground water pressures behind the Guard Wall. Design water level for the wall is elevation 40 feet. This level is about 32 feet below the adjacent river level. Based on evaluation of the test drain program and the success of similar drains the Tanner Creek Landslide, a system of horizontal drains was selected. To lower the ground water level below the river level by gravity drainage, the horizontal drains were designed to be installed from within drainage shafts. Since the Guard Wall was constructed by slurry methods through the landslide debris to undisturbed bedrock, it cuts off potential seepage from the river.

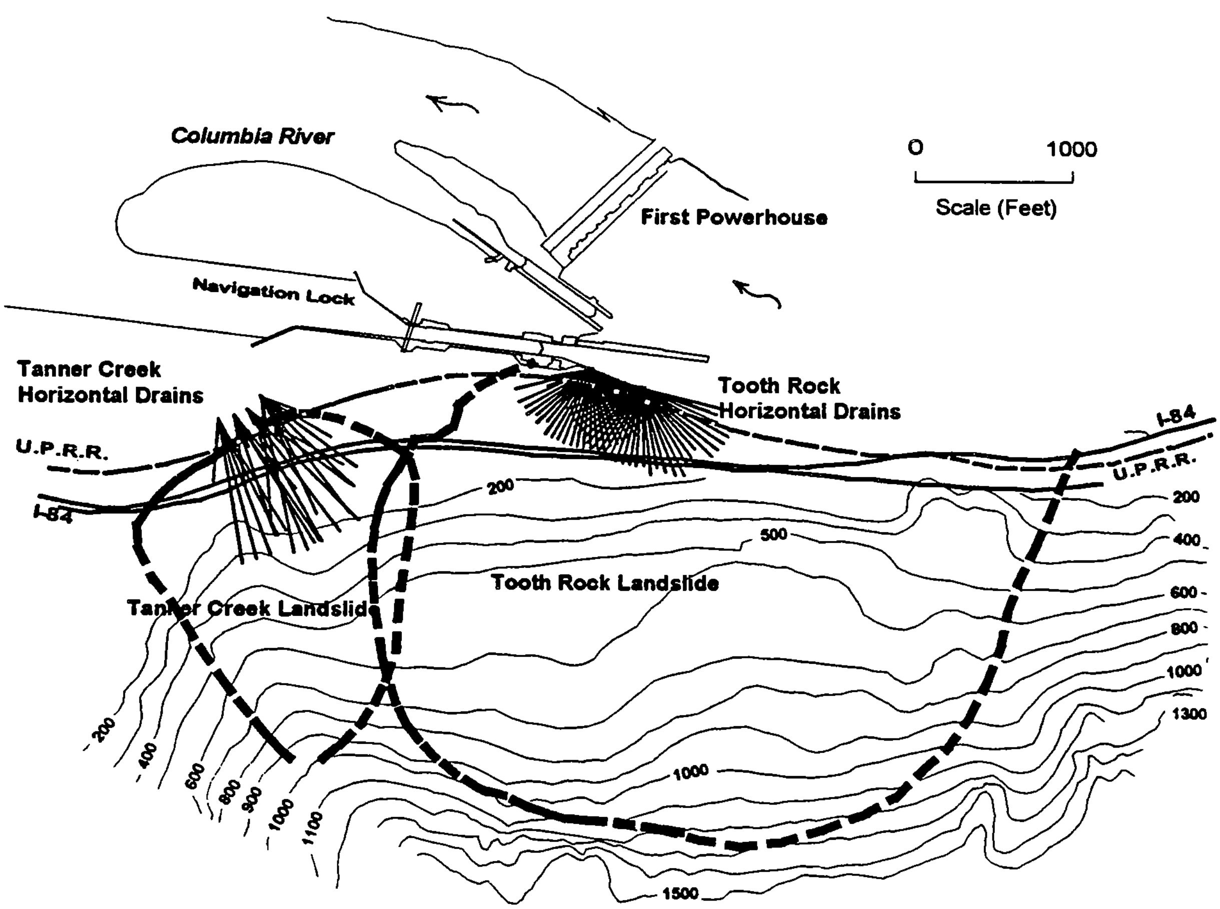
The horizontal drain system was sized by estimating the amount of runoff expected from the normal infiltration per year, plus a 25 year surge. Drainage of the mass is considered to take place over a 9 month period and is assumed to be 75 percent effective. Based on the design information, the following were estimated:

	Cubic Feet	Gallons
Normal Infiltration Per Year	38 million cubic feet	287 million gallons
25 Year Surge	1.6 million cubic feet	12 million gallons
Designed Infiltration	40 million cubic feet	299 million gallons
Estimated Flow Per Drain		15.5 gallons per minute
Estimated Number Of Drains Required	51 Drains	
Estimated Number Drains Required Operating At 60 Percent Efficiency	85 Drains	

Based on the number of drains and estimated flows per drain, a peak flow of about 790 gallons per minute was estimated.

Construction of Tooth Rock Landslide Drains

The Tooth Rock drainage system as constructed is shown on Figure 4. The shafts are designated as Shaft No. 2 and 3. The system was originally designed with a third shaft and array of drains. However, cost evaluations during construction determined that a drainage system utilizing two shafts with longer horizontal drains would save about $480,000 (RZA AGRA, 1992). The eastern most shaft (Shaft No. 1) was therefore eliminated and drains from Shaft No. 2 were extended to the southeast.



TOOTH ROCK AND TANNER CREEK
HORIZONTAL DRAINS
FIGURE 4

Shafts No. 2 and 3 are 14-foot inside diameter constructed to depths of 81 and 52 feet, respectively. The two drainage shafts are interconnected with a four foot diameter concrete lined tunnel. Because of the variable compostion of the landslide debris, the interconnecting tunnel was completed using a combination of hand mining techniques and hydraulic jacking. Collected water from the downstream shaft is, in turn, discharged through a 20-inch diameter pipe downstream. The entire drainage system operates by gravity flow.

The drainage shafts permit drilling arrays of horizontal drains from multiple levels below the ground surface. One hundred twenty three drains were drilled for a total of 56,005 lineal feet. Maximum length of production drains was approximately 600 feet. Most drains were designed to be inclined upward from the horizontal at a 5 degree slope. However, some drains that parallel the wall were inclined 2.5 degrees. Directional drilling techniques were not used.

Each drain consisted of a 4-⅛-inch diameter drilled hole cased to the full depth. A permanent 1½-inch diameter slotted PVC liner was installed and the casing removed. The 20 feet of pipe nearest each drainage shaft is non-slotted PVC pipe with the first ten feet of the blank section nearest the shaft grouted in-place. Control of the direction of drilling was a significant problem. Because of the rotation used in the drilling process, drain holes tend to veer up and to the right. To compensate, the drains were started as much as five degrees from planned inclination (in some cases with a negative inclination relative to the horizontal) to compensate for this tendency.

Hydrogeologic Performance and Evaluation of Tooth Rock Landslide Drainage

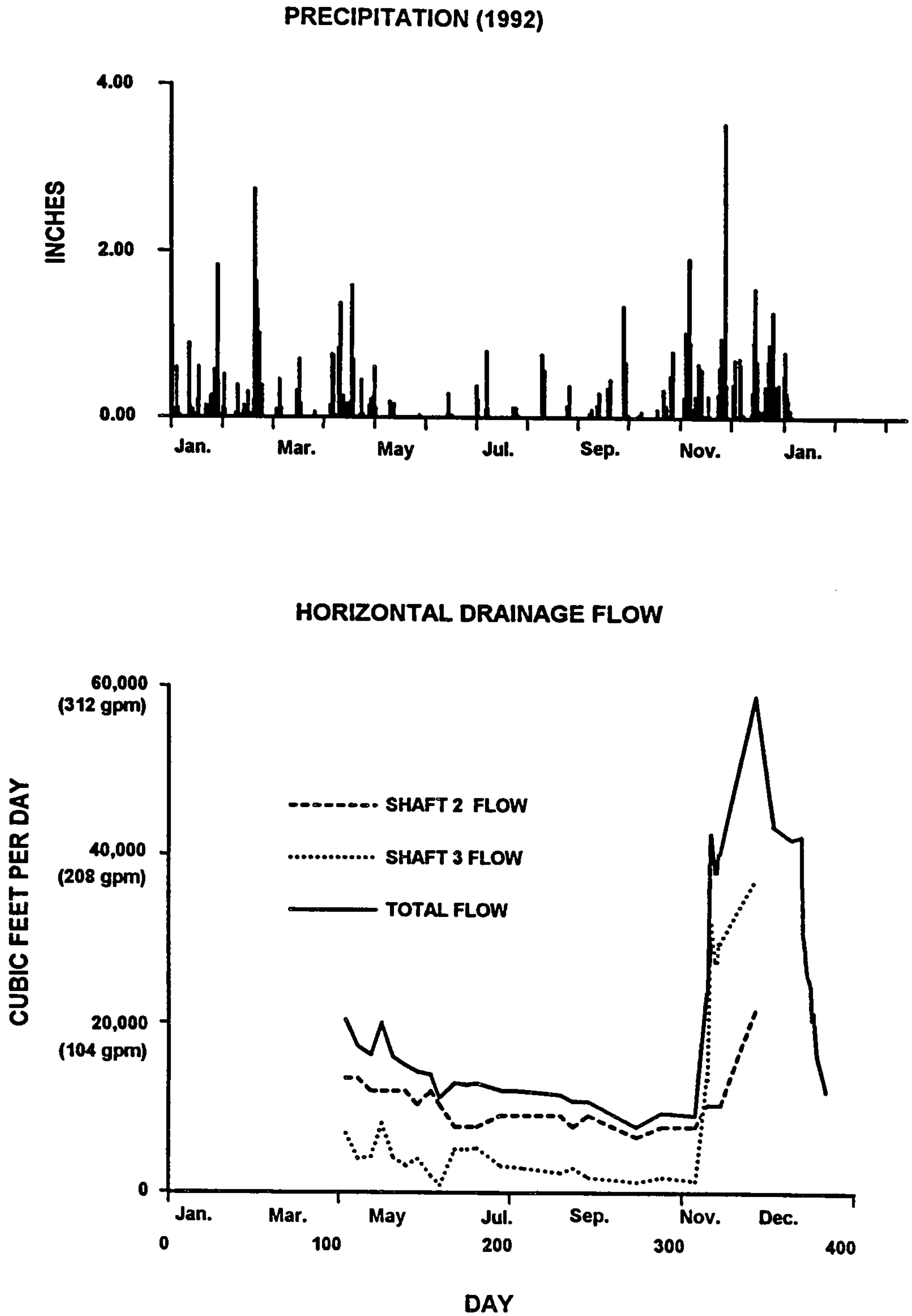
The drainage system is being monitored: it has functioned well. Piezometers have shown that the ground water levels, thus, hydrostatic pressures have been reduced against the Guard Wall below the design level. The measured drainage flow from the system of drains along with the precipitation for 1992 is shown on Figure 5. The zero date is January 1, 1992, when the automated flow measuring system was completed. The figure shows that the total flow decreased from about 20,000 to 6,000 cubic feet per day (80 to 30 gallons per minute). This decrease follows what would be expected as a recession curve or base flow. Integrating this recession flow suggests that over a period of a year, about 5 million cubic feet (37 million gallons) of water would drain.

A winter peak flows of about 300 gallons per minute was observed. About 200 to 250 gallons per minute of this flow represents increase infiltration due to winter precipitation. The partially saturated landslide debris above the ground water table appears to moderate the infiltration flow. Consequently, there are no sharp discharge peaks related to individual storm events and the drainage out flow is steadier than the variable input precipitation. Based on the estimated quantity of water above the summer base flow and the amount of rainfall, it appears that about 27 acres land surface was directly contributing water. This is nearly twice the area covered by the horizontal drains.

Limited available data suggest that total drainage has been about 8 million cubic feet (60 million gallons) of water. This quantity of water is below the initial estimate based on a larger contributing area.

SUMMARY

The drainage system installed in Tooth Rock Landslide is performing well in lowering the water levels and hydrostatic pressures against the Guard Wall located at the toe of the landslide. The total quantity of water is less than would have been predicted by design. However, the rate at which water infiltrates and is drained during the winter months may be higher than assumed in the design. The exploration program and design effort resulted in a conservative design which functions satisfactory.



TOOTH ROCK DRAINAGE

FIGURE 5

REFERENCES

Beaulieu, J.D., 1977, *Geologic Hazards of Parts of Northern Hood River, Wasco, and Sherman Counties, Oregon*: Oregon Department of Geology and Mineral Industries Bulletin 91, 95p, 3 sheets.

Hammond, P.E., 1980, *Reconnaissance Geologic Map and Cross Sections of the Southern Washington Cascade Range*: Department of Earth Sciences, Portland State University, Portland, OR, 31 p., 2 sheets.

RZA AGRA, Inc., 1992, Diaphragm Walls Foundation Construction Report, Main Navigation Lock, Bonneville Dam: Prepared for U.S. Army Corps of Engineers, Portland District, Portland, Oregon.

Sager, J.W., 1989, Bonneville Dam: *Engineering Geology in Washington, Vol. 1*: Washington Division of Geology and Earth Resources Bulletin 78, pp. 337-346.

USCOE, 1984, *Geology, Excavation and Foundation Design Memorandum No. 3, Bonneville Navigation Lock, Columbia River, Oregon/Washington*: Portland District U.S. Army Corps of Engineers, Portland District, Portland, OR.

USCOE, 1988, *Geology, Excavation and Foundation Design Memorandum No. 3, Supplement No. 1, Bonneville Navigation Lock, Columbia River, Oregon/Washington*: Portland District U.S. Army Corps of Engineers, Portland District, Portland, OR.

Wise, W.S., 1970, *Cenozoic Volcanism in the Cascade Mountains of Southern Washington*: Washington Division of Mines and Geology Bulletin 60, Olympia, WA, 45 p., 1 plate.

RECENT GEOTECHNICAL ADVANCES IN THE DESIGN AND CONSTRUCTION OF HIGHWAY TUNNELS

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ABSTRACT

There have been a number of advances in the design and construction of highway tunnels in recent years. The majority of these have come directly from the geotechnical arena and have resulted from our improved understanding of ground behavior during tunneling, the support required to maintain a stable underground opening, improved excavation techniques, and improved contracting practices for tunnel construction. For tunnel projects to be successful, it is essential that the geotechnical engineers and geologists on the project team play a pivotal role because the highest costs and greatest risks of a tunneling project are nearly always governed by geologic and geotechnical considerations.

This paper describes some of the recent developments in geotechnical engineering for highway tunnels, including tunnel investigations, tunnel layout, ground support, excavation methods, waterproof lining design and construction, and contracting practices. To illustrate these developments the paper describes a number of case histories of recent tunneling projects in the south and east USA.

INTRODUCTION

The key elements of most highway tunneling projects are the geologic and geotechnical conditions, the investigation techniques, the alignment and roadway design, the excavation methods, the support design and installation, the tunnel lining, the drainage, the lighting, the ventilation, and the contracting practices. It is essential that the design and construction of a tunnel project are not divided up between different departments and different technical disciplines, each of which acts independently and only sees their piece of the puzzle. The tunneling team must act as a cohesive unit with all disciplines and individuals working together. It is highly desirable that the design and construction team be lead by someone who has a good basic understanding of geology and ground behavior during tunnel excavation, particularly during the planning and early design of the project.

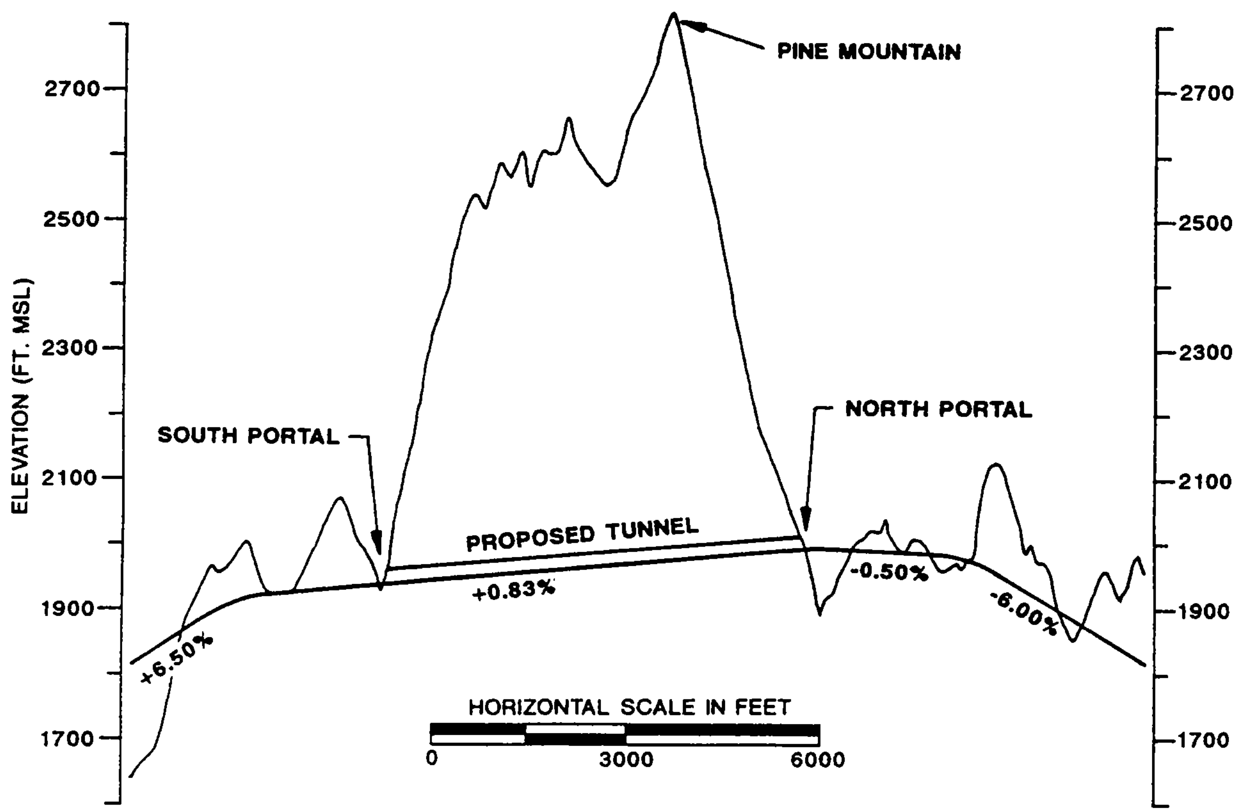
A lot of publicity has been given to the New Austrian Tunneling Method, or NATM, in recent years. However, there is no single, accepted definition of NATM and even Austrians cannot agree on its definition. Generally, NATM is used to describe a method of tunnel design and excavation which involves observing and monitoring the behavior of the rock as a tunnel is excavated, installing an appropriate level of active support to get the rock to support itself, and using rockbolts, shotcrete, and lattice girders as the ground support elements. This method of tunneling is also called the observational method, the design-as-you-go method, and the positive support method. It has been used widely in the United States for the last 15 years and NATM can now also be used to describe the North American Tunneling Method.

While much publicity has been given to NATM, the greatest advance in tunneling technology in the last 20 years has been the development of efficient tunnel boring machines (TBMs). TBMs are now widely used and are capable of very rapid excavation at low cost in the right ground conditions. However, TBMs are not applicable in all conditions as is described later in this paper.

TUNNEL LAYOUT AND SIZE

Most modern highway tunnels require three, four or more traffic lanes for the projected volume of traffic and for passing lanes. However, it is usually more expensive to construct one large three- or four-lane tunnel than to construct two two-lane tunnels, because of the volume of rock that needs to be excavated to form an arched tunnel crown and because proportionally more support is required for wider tunnel spans. Consequently, the geotechnical engineers and geologists must work closely with the alignment and roadway engineers to adjust the grades and number of traffic lanes to provide the most economical configuration.

An example of this is at the proposed Pine Mountain Tunnel for US 119 in Southeastern Kentucky, where the dipping sedimentary sequences vary in quality from moderate to poor. It was considered that three-lane tunnel would be difficult and expensive to construct in the poorer rock units. However, one-lane in each direction plus a passing lane are required for steep grades. To avoid having to construct twin tunnels, a significant effort was expended by the tunnel design team to find an alignment where the grades on the tunnel and portal approaches are flat enough to not need passing lanes. A profile of the selected alignment and the preliminary design of the tunnel are shown on Figures 1 and 2.

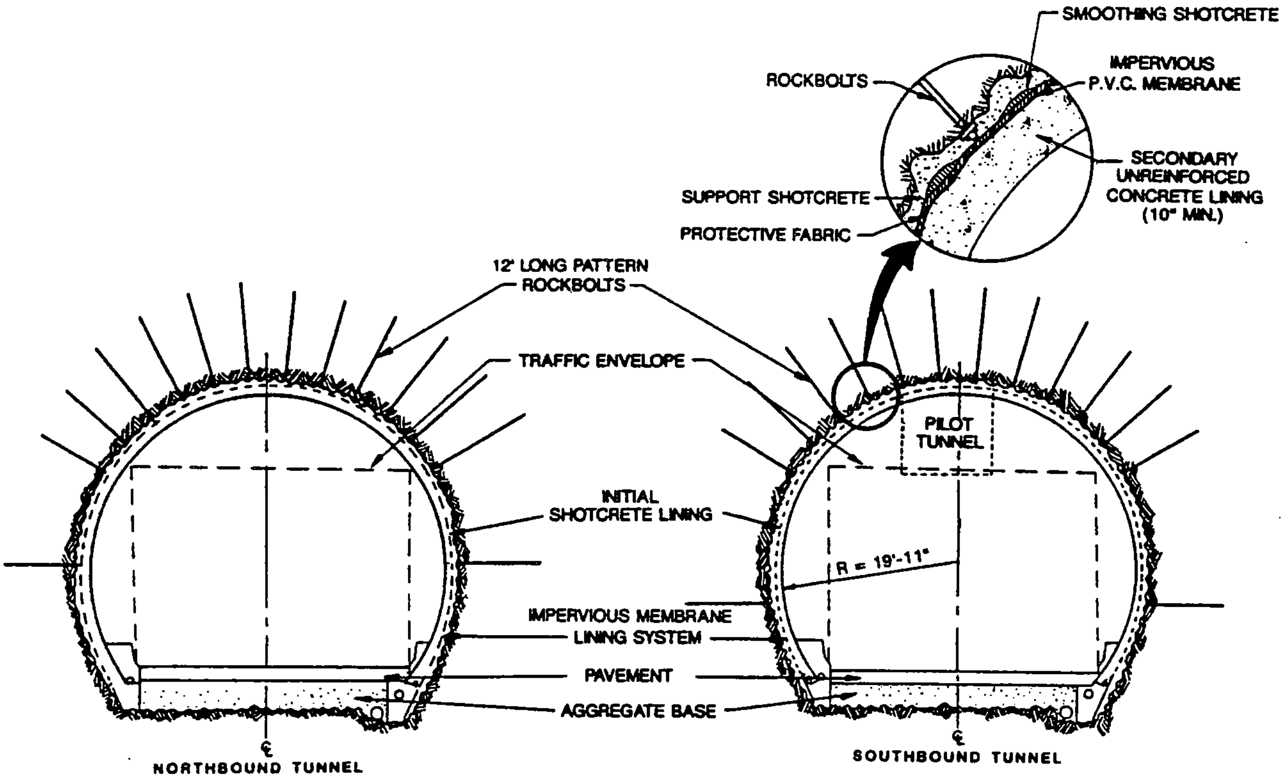


PROFILE OF PROPOSED PINE MOUNTAIN TUNNEL

FIGURE 1

The geology at the Cumberland Gap Tunnels on US25E between Tennessee and Kentucky is very similar to the geology at Pine Mountain. However, the volume of traffic requires two-lanes of traffic in each direction. Figure 2 shows the shape, dimensions and spacing of the two twin-tunnel configuration that was constructed.

In general, twin tunnels are preferable where traffic volumes justify four lanes of traffic as the piston effect of the traffic improves ventilation. In addition, public safety is improved as cross passages can be provided at regular intervals between the tunnels to give motorists pedestrian access to the other tunnel in event of a tunnel fire.



CROSS SECTION THROUGH CUMBERLAND GAP TUNNELS

FIGURE 2

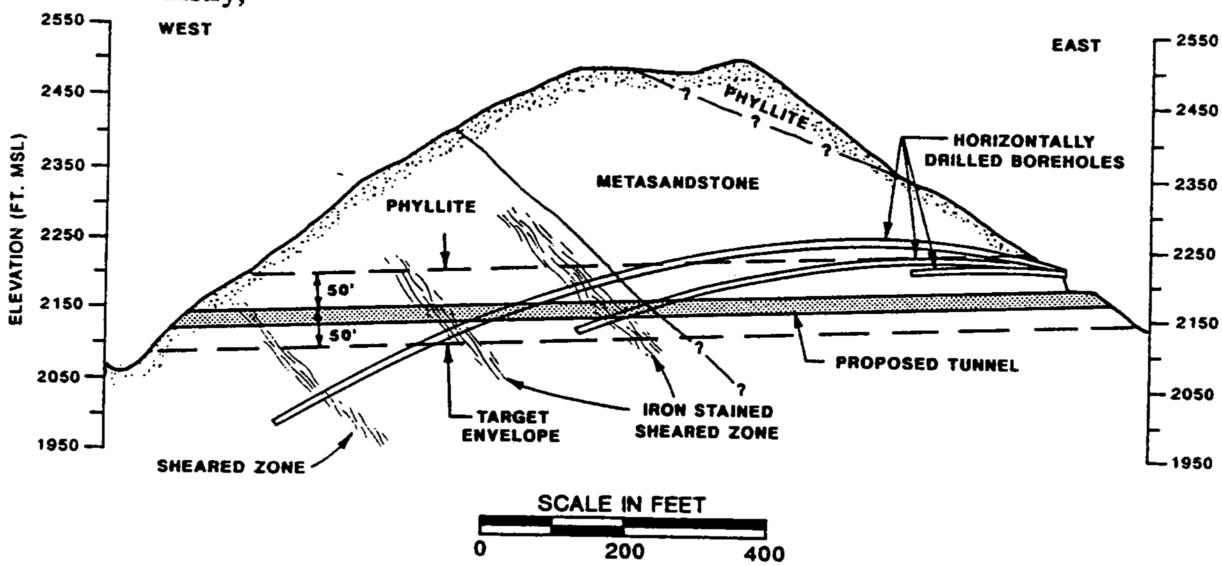
TUNNEL INVESTIGATIONS

The quality of data that is available for the design of tunnels has improved remarkably in recent years. This has been the result of improved geotechnical investigation techniques which have provided a better understanding of the ground conditions that will be encountered during tunnel excavation. This enables tunnel engineers to design appropriate levels of ground support and specify the most appropriate method of tunnel construction.

These improved geotechnical investigation techniques include the following:

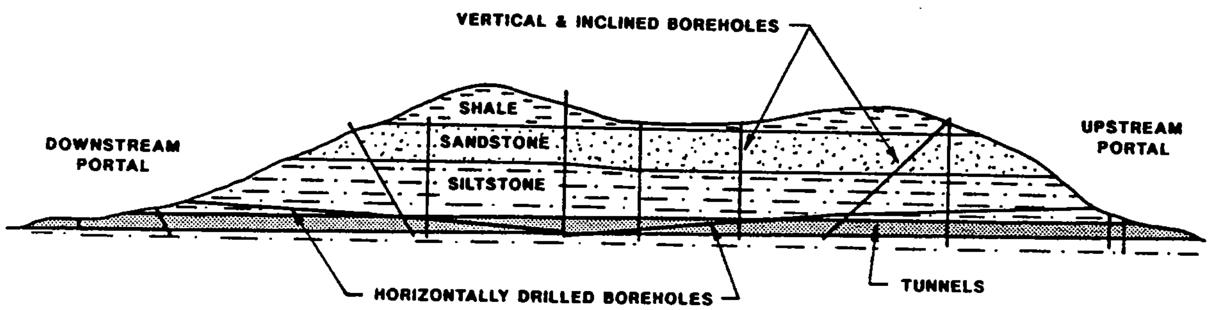
- The use of split inner barrels for rock coring which provides good data on the roughness, infilling, spacing, and wall strength of discontinuities as well as on the pieces of rock core;

- Horizontal drilling techniques which are more economic and more effective at investigating the rock conditions along the axis of the tunnel than boreholes drilled from the surface. Two examples of this horizontal coring technique are shown on Figures 3 and 4. Figure 3 shows the horizontal coring that was done along the axis of the proposed Crooked Arm Ridge tunnel on the Foothills Parkway in Tennessee (Reference 1). At this site the drilling contractor elected not to redirect the drill holes. Instead he started coring three times to obtain sufficient data on the rock along the alignment. Figure 4 shows the horizontal coring that was done to investigate the Harlan Flood Diversion Tunnels in southeastern Kentucky (Reference 2) where the core holes remained remarkably straight and level without redirecting because of the horizontal bedding of the siltstone unit. The cost of horizontal coring is usually two to three times the cost of vertical coring. This includes redirecting the drill bit so that it remains within a target envelope. The usual limit of drilling length is about 2,000 ft. though longer holes are common in the mining industry;



GEOLOGIC PROFILE & INVESTIGATIONS OF PROPOSED CROOKED ARM RIDGE TUNNEL

FIGURE 3



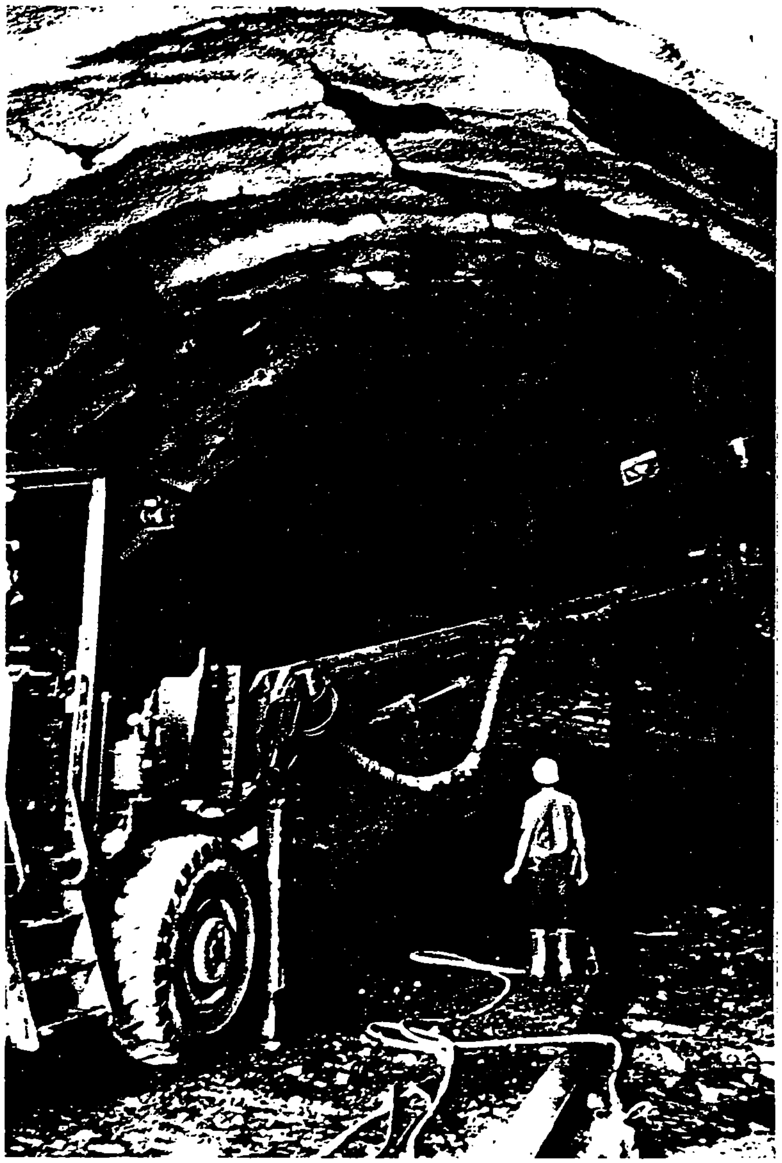
GEOLOGIC PROFILE & INVESTIGATIONS OF HARLAN TUNNELS

FIGURE 4

- The use of modern hydrogeologic investigation techniques which include pumping tests, triple packer permeability tests using downhole pressure transducers, geochemical analyses of the groundwater, and sophisticated hydrogeologic modeling;

- The use of sophisticated geophysical methods which include ground penetrating radar, cross hole seismic refraction, and resistivity, and gamma logging of boreholes; and

- The use of an exploratory pilot tunnel along the axis of the main tunnels to investigate particular or difficult geologic conditions. A good example of the successful use of a pilot tunnel is at the Cumberland Gap Tunnels (Reference 3). The pilot tunnel in the face of the main southbound tunnel is shown on Photograph 1 and Figure 2.



PHOTOGRAPH 1

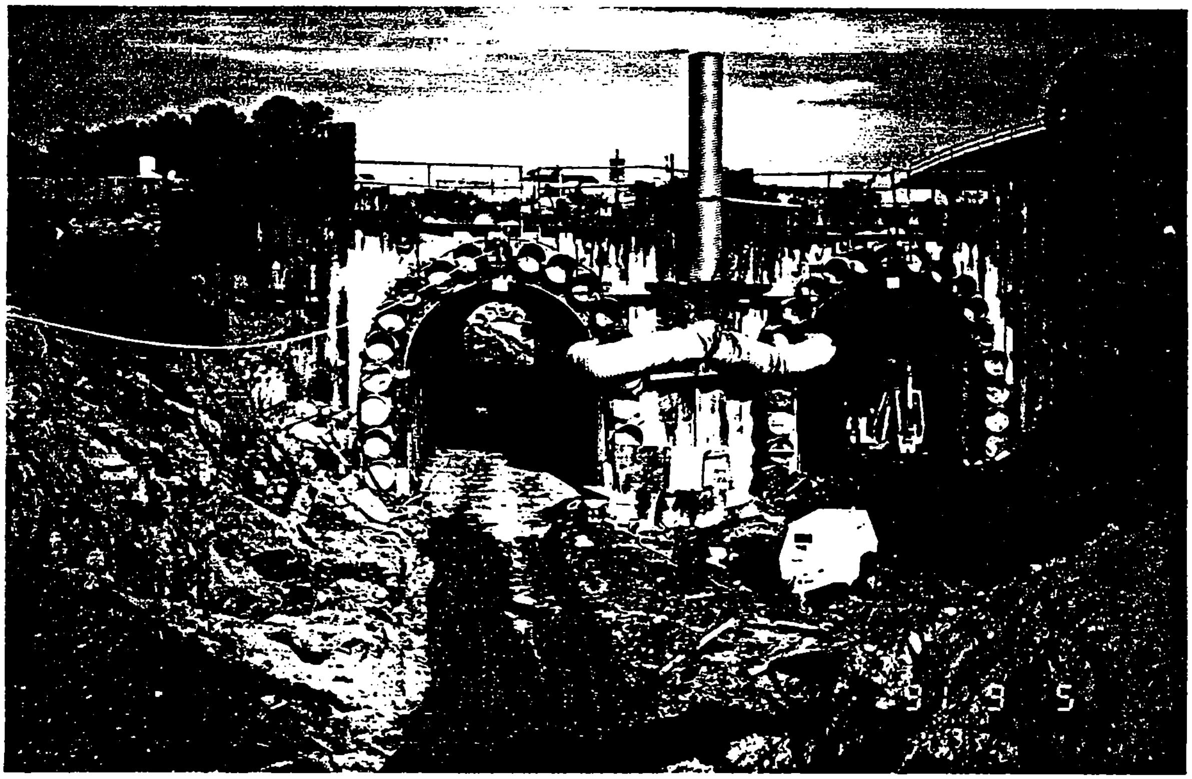
Drilling the face of the top heading of the southbound tunnel at Cumberland Gap. The pilot tunnel can be seen in the face at crown level.

GROUND BEHAVIOR AND SUPPORT DESIGN

In the last 20 years, significant advances have been made in the understanding of the inter-relationship between tunnel excavation methods, tunnel displacements, tunnel support types, and timing of support installation. Most highway tunnels are at relatively shallow depths below the ground surface. Where the tunnels are in rock, the behavior of the ground is normally controlled by the jointing, bedding planes, shear zones, and other discontinuities in the rock structure rather than by the stresses induced in the rock surrounding the tunnels. In these circumstances, the behavior of the ground is called structurally controlled and the objectives of installing ground support are to limit or control the ground displacements, to minimize the loosening of the rock mass surrounding the tunnel, and to knit the rock mass together so that it maintains its inter-locked, intact strength. The most effective way of doing this is to install rockbolts and shotcrete in good rock and shotcrete and lattice girders in poorer rock close to the excavation heading. As discussed earlier, this method of support is often known as NATM and is now widely and routinely used in North America.

In soft ground conditions, where the tunnel is in soil or soft rock, the primary objective of the tunnel support is to minimize ground settlements above the tunnel and to prevent lowering of the water table. There are a wide range of techniques that are now available for use in particular site conditions, and significant advances have been made recently in the techniques of jet grouting, earth pressure balance TBMs, and micro-tunneling to form an arch around the perimeter of the future tunnel.

An example of the use of jet grouting and micro-tunnels around the perimeter of the tunnel is shown on Photograph 2. This shows the Metro Atlanta Rapid Transit Authority (MARTA) light rail transit tunnel under Interstate 285 in Atlanta. At this site, there was only eight feet of cover between the crown of the tunnel and the 12-lane active interstate ring road around Atlanta. The interlocking micro-tunnels were bored first to provide continuous ring support around the perimeter of the main tunnels. The fill and saprolite soils inside the micro-tunnel ring were then jet-grouted prior to excavation so that the face of the excavation would stand steeply to enable the final arch ring support to be installed close to the excavated heading.



PHOTOGRAPH 2

MARTA tunnels are under I-285 in Atlanta. The interlocking microtunnels were bored prior to main tunnel excavation.

One of the key support elements of modern hard and soft ground tunnels is shotcrete. There have been remarkable advances in pneumatically applied concrete technology and machinery that is used in modern shotcreting. These include the following:

- The use of steel fibers or polypropylene fibers in the shotcrete mix in place of wire mesh reinforcing. Fiber reinforced shotcrete can have better bending resistance, toughness and punching shear resistance than mesh reinforced shotcrete. In addition, it can be placed in one application by remote shotcrete jumbos immediately after excavation rather than having to place wire mesh against the rock and then later shoot the shotcrete. In addition, there is less wastage of shotcrete to fill depressions behind the wire mesh;

- Silica fume additives for shotcrete are widely used as they reduce rebound, increase density, increase durability, and enables greater thicknesses of shotcrete to be applied in one shooting;

- Wet mix and dry mix shotcrete technology have both been developed to where they can both provide satisfactory results. The most economic of these methods can be selected to suit the remoteness of application, the quantity used in each application, and the access to the site; and

- Improvements in the design of shotcrete pumps, shotcrete hoses, nozzles, and shotcrete remote robot arms. It is now possible to place shotcrete at high production rates, at consistent quality and in poor ground in the tunnel crown where it is unsafe for miners to work.

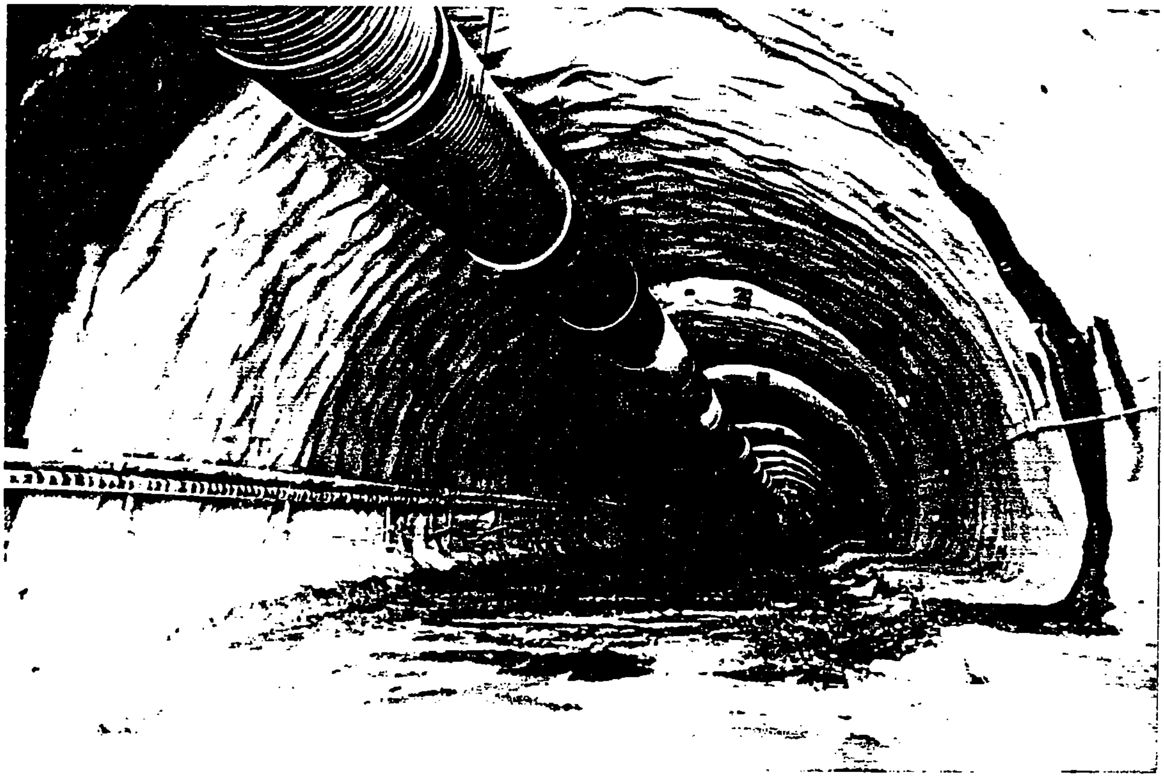
EXCAVATION METHODS

Probably the greatest single advance that has been made in tunneling technology in recent years is the development of efficient TBMs. These machines have significantly decreased tunneling costs and increased the rates of excavation. The improvements are on-going and every year new records seem to be achieved for tunnel advance rates. TBMs can now be used for a wide variety of ground conditions, from soft clay to very hard rocks, though each TBM is made for a limited range of ground conditions. TMBs can also now be made in a wide range of sizes from micro-tunnels which are less than two feet in diameter up to large tunnels which are 40 feet or more in diameter. A TBM itself constitutes a large upfront cost for any tunneling project, so the use of TBM excavation is often not justifiable for short highway tunnels. The minimum economic length for which a TBM is usually considered is between one and two miles. There are now many used TBMs that can be refurbished and used for shorter tunnels to reduce up front costs and cut down an initial delivery time.

One of the main disadvantages of TBMs is that any individual TBM can only work effectively within a limited range of ground conditions. Consequently, when the ground conditions vary rapidly or where a hard rock tunnel passes through zones of squeezing ground or a fault zones, TBMs often become stuck. Consequently, TBMs are usually only used on the largest highway tunnel projects which are in uniform ground conditions.

For shorter tunnels in soft and medium strength rock, roadheaders are often the most economical method of excavation. Like TBMs, roadheaders provide considerably less disturbance to the ground surrounding the tunnel than the conventional drill-and-blast method, and, consequently, support costs can be reduced somewhat by the use of roadheaders. An excellent of the example of the use of a roadheader was in the recently completed Harlan Flood Diversion Tunnels in southeastern Kentucky. Photograph 3 shows the smooth profile of one of the shotcrete lined tunnels which was excavated by a 120 ton roadheader in the 6,000 psi siltstone units at Harlan.

For many highway tunnels in rock, the most common and often the cheapest and most flexible method of excavation is still the conventional drill-and-blast method, which is shown in Photograph 1. There have also been significant advances in drilling and blasting techniques in recent years which improve production rates, decrease costs, and reduce overbreak and the damage to the rocks surrounding the tunnels. This method remains the most flexible method of excavation for ground conditions which may vary significantly over short distances.



PHOTOGRAPH 3

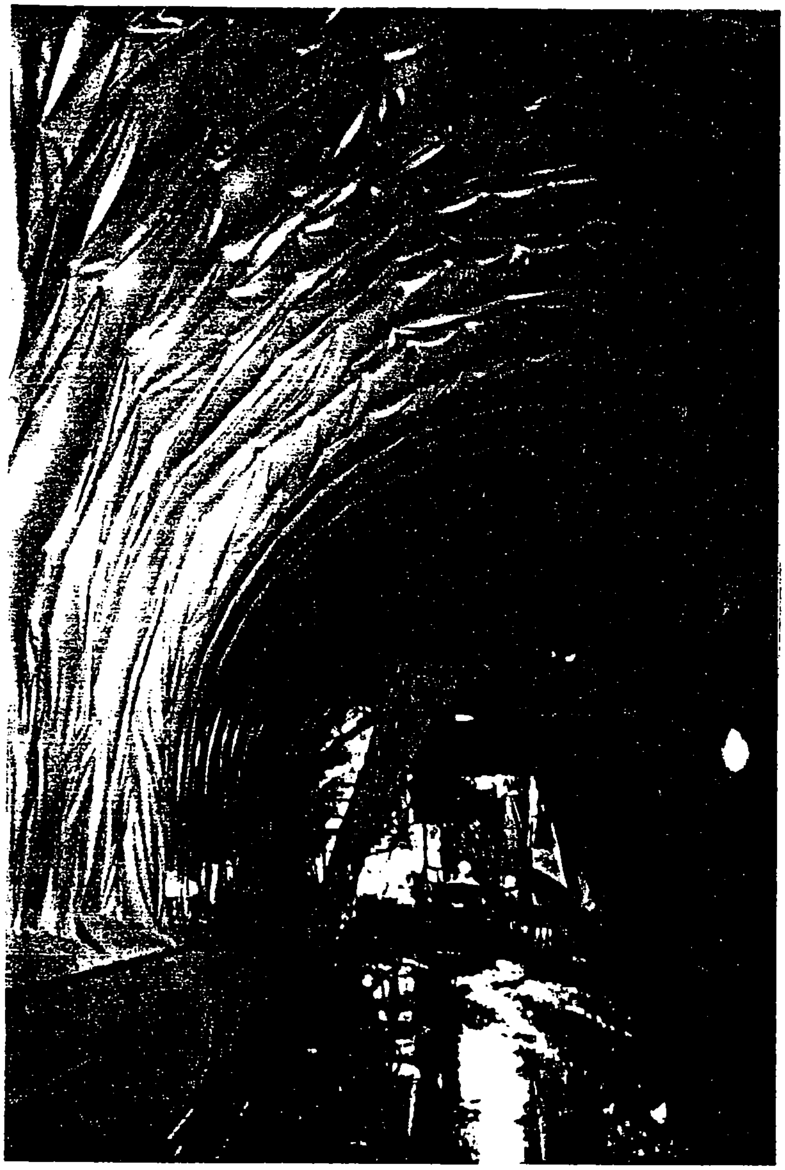
Harlan Tunnels in Kentucky excavated by roadheader. Rockbolts and shotcrete provide the final support and lining.

TUNNEL LINING

Several recent studies have shown that groundwater flows into tunnels significantly increase the rate of deterioration of the tunnels and increase tunnel maintenance. The use of a geomembrane lining has become very popular in the last five years. This lining system is shown on Figure 2 and Photograph 4. The main elements of the system are as follows:

- Rockbolts, shotcrete, and lattice girdles provide the excavation support and long-term ground support;
- A layer of smoothing shotcrete reduces the irregularities in the shotcreted profile;
- A geotextile protection fabric acts as a drainage layer and cushion for the PVC membrane. This is attached directly to the shotcrete;
- An impervious PVC membrane which is attached to the geotextile fabric and seamed in place; and
- Interior concrete arch lining to support the PVC lining. This is required to have sufficient strength to withstand external groundwater pressure loading on the PVC lining.

With this tunnel lining system, drainage is usually provided at the invert of the tunnel so that the lining system does not have to withstand the full external groundwater pressures. However, there will be some buildup of external water pressure on the PVC membrane so the most economical shape for the tunnel is a circle or an arch shape which minimizes the bending moments in the concrete arch lining.



PHOTOGRAPH 4

PVC waterproof membrane prior to concrete arch construction at Cumberland Gap.

TUNNEL CONTRACTING PRACTICE

A very encouraging development in tunnel construction is the improved tunnel contracting practices that has been adopted in recent years. The US National Committee on Tunneling Technology has brought out several recent publications giving recommendations for improved tunnel contracting practice (References 4 and 5). These have been widely adopted by the tunneling industry and, consequently, the number of tunnel construction projects that have gone to court has sharply reduced. It is now generally accepted that the risks have to be shared between the owner and the contractor, that the owner owns the ground and is therefore responsible for paying for the support that is required to stabilize the tunnel, and that the contractor is responsible for construction safety. The other changes that have been brought about by this enlightened contracting practice include the following:

- Full disclosure of all data from investigations and testing that have been performed by the owner;

- The use of flexible, Unit Price Contracts, where the Contractor gets paid for the rock support that is required and the ground type that is encountered, rather than Lump Sum Contracts;

- The presence of experienced geotechnical personnel on site during construction who are able to adapt the support design to suit the ground conditions as they are encountered during tunnel excavation;

- The use of geological and geotechnical mapping of the exposed rock in the tunnels to assist with the design of the support of the tunnels;

- The use of a Disputes Review Board which has one member selected by the contractor, one member selected by the owner and the chairman selected by the first two members. The board meets regularly during construction and hears both sides of potential disputes before they develop into claims;

- The requirements for the successful bidder to place his bid documents in escrow. The documents can only be retrieved by both parties in the event of a dispute; and

- The use of a Geotechnical Design Summary Report (GDSR) which forms part of the bid documents and presents the engineer's interpretation of the ground conditions that are expected and the reaction that is expected of the ground to the tunneling process.

REFERENCES

1. Humphries, R.W. and Ingram, P.M., "Investigations and Preliminary Design of Crooked Arm Ridge Tunnel", Southeastern Transportation Geotechnical Conference, Gatlinburg, TN, October 1985.

2. Simmons, M.D. and Stanton, J.L., "Harlan, Kentucky, Diversion Tunnels", Bulletin of the Association of Engineering Geologists Vol. XXVIII, No. 1, 1991.

3. Humphries, R.W., Sullivan, W.R., and Leary, R.M., "Instrumentation of Cumberland Gap Pilot Tunnel", TRB 69th Meeting, Washignton, D.C., January 1990.

4. U.S. National Committee on Tunneling Technology, National Academy of Science, "Better Contracting for Undergound Construction", Washington, D.C., 1974.

5. Technical Committee on Contracting Practice of the Underground Technology Research Council, "Avoiding and Resolving Disputes in Underground Construction", 1989.

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"HAZARDOUS MATERIALS IN THE ROADWAY"

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Transportation Department

ABSTRACT

The presence of hazardous materials within the roadway corridor can impact Departments of Transportation (DOT's) through delays in construction activities, create roadway construction worker health and safety concerns, increase the DOT's liability in Right of Way acquisitions, and cause delays in project planning. The New Mexico State Highway and Transportation Department (NMSHTD), headquartered in Santa Fe, New Mexico, has taken a proactive approach to the hazardous materials issue by establishing guidance and procedures which are documented in a "Handbook of Hazardous Waste Management". The procedures established in this guidance document are the result of situations encountered by the NMSHTD in project construction, most notably in Hatch, New Mexico in 1990.

The Hatch Project consisted of the reconstruction of State Road SR-26 through a small farming community in south-central New Mexico. Construction activities included installation of a forced main storm sewer, sidewalk improvements, intersection reconstruction, and other roadway improvements. A gasoline service station, which utilized underground storage tanks to dispense fuels, was acquired in a ROW condemnation action for intersection improvements. During trenching activities for the forced main storm sewer, impacts to soils and groundwater from petroleum fuels were encountered. The fuel contamination investigation was required to comply with state environmental regulations, modifications to proposed construction activities, and subsequent litigation which delayed project completion by over one year. Project costs increased by approximately one million dollars. Members of the NMSHTD Geology Section were called upon to provide assistance in investigation, litigation support, and project re-design. The NMSHTD subsequently modified its procedures and established policies to avoid future problems.

Highway geologists are frequently the first to become involved in hazardous materials investigations, being perhaps the best qualified among DOT professionals to guide their agencies through these, sometimes new, confusing issues. Highway geologists may be involved in, or provide technical oversight of, the investigation of these facilities. Working closely with the DOT's Office of General Counsel, a pro-active approach to hazardous materials should include environmental audits during the preliminary design phase to determine if potentially impacted properties are located within the roadway corridors. Utilizing their knowledge of geology, geo-hydrology, and aqueous chemistry, highway geologists can assist in the determination of the nature and extent of impacts to roadway construction projects. Prior planning in project design is absolutely necessary to avoid liabilities which can decimate project budgets and schedules.

INTRODUCTION

The New Mexico State Highway and Transportation Department (NMSHTD), like every Department of Transportation (DOT) in the nation, is charged with the construction, maintenance and improvement of the public roadway system. Acquisition of Right-of-Way (ROW), construction projects, and maintenance activities have, increasingly, become affected by the proliferation of laws, regulations and public awareness. Most state DOT's have had experiences relating to the presence of what are commonly known as hazardous wastes in the roadway corridor. These experiences are generally costly and time consuming and have prompted many DOT's to take a proactive approach toward recognition of potentially impacted properties.

Promulgation of environmental legislation in the 1970's included the landmark Resource Conservation and Recovery Act (RCRA) of 1976 and the Comprehensive Environmental Response, Compensation and Liability Act (CERCLA) of 1980. The Hazardous and Solid Waste Amendments (HSWA) of 1984 amended RCRA, and the Superfund Amendments and Reauthorization Act (SARA) amended CERCLA in 1986. Additional landmark legislation included the Toxic Substance Control Act (TSCA), and the Federal Insecticide, Fungicide and Rodenticide Act (FIFRA). This climate of stricter environmental regulations and increased public scrutiny, environmental audits and site assessments have become valuable tools for regulatory compliance and environmental risk management.

Two studies were conducted at the request of the American Association of State Highway Officials (AASHTO) acting through the National Cooperative Highway Research Program (NCHRP). The Transportation Research Board (TRB) convened a committee of 14 experts to examine these problems. The result of their work first appeared as NCHRP Report 310, "Dealing with Hazardous Waste Sites", with follow up research (NCHRP Project 20-28) resulting in NMCHRP Report 351, "Hazardous Wastes in Highway Rights-of-Way". Report 351 documents the findings of the committee as a result of numerous interviews with experts in the field as well as interviews with 16 state DOT's. Case studies abound in the field of hazardous wastes in roadway corridors, and Report 351 also presents several.

The principal findings of the committee are given as:

"(1) ...hazardous wastes are frequently encountered and are potentially present in nearly all DOT projects;

(2) Hazardous wastes can present serious liabilities to DOT's in terms of cost, delays, and threats to the health and safety of both employees and the public;

(3) hazardous waste problems are manageable with procedures and approaches available to DOT's for developing hazardous waste programs;

(4) petroleum-related contamination is the most commonly encountered problem but is one for which relatively well-developed procedures are available;

(5) early detection of hazardous waste is important to maximize the options available to DOT's and permit sound business decisions concerning it;

(6) the relationship between DOT's and their state environmental regulatory agency (SRA) can be very important to a successful hazardous waste program;

(7) solutions to the problems of appraisal of contaminated properties and cost recovery are still evolving; and

(8) groundwater contamination presents a potential long-run problem for DOT's."

The lessons learned by the NCHRP committee through interviews with state DOT's and the guidance offered by the consulted experts indicate the magnitude of the problem facing the DOT. The Federal Highway Administration developed a student course manual entitled "Hazardous Waste: Impacts on Highway Project Development", to be used in a National Highway Institute (NHI) training course on that subject.

Among the most common problems relating to hazardous materials encountered by DOT's are lead paint, solvents and pesticides, salt (from roadway de-icing activities), and Underground Storage Tanks (UST's). Internal generation of potentially hazardous materials (such as roadway salts and lead paint from bridge re-habilitation) and abandoned or orphan hazardous materials left along the roadway also are problems for the DOT, but beyond the scope of this discussion.

The acquisition of Right-of-Way presents a continuing source of problems to the DOT in the area of hazardous materials. Highway geologists are probably the best equipped to steer our agency through these difficult problems and decisions. The geologists' knowledge of hydrogeology, soils and fate and transport of contaminants, along with the DOT's access to drilling equipment frequently places them in a position to guide their Department. A case study of one such occurrence is presented to illustrate New Mexico's experience and set the stage for a solution to the problem.

THE HATCH EXPERIENCE

For the NMSHTD, the awakening came in November, 1990, in the small farming community of Hatch, New Mexico. Hatch, which bills itself as the Chile Capital of the World, lies in south-central New Mexico in the valley of the Rio Grande, in the Palomas basin, extending from San Diego Mountain, the Sierra de las Uvas, and the Goodsight Mountains northward. In the area of the Rincon Valley, sediments are derived from both nearby locations and from sources upstream, Red clay beds underlying the alluvial material are indicative of lacustrine deposition typical of a closed basin and are up to two thousand feet thick. These red clay beds are found locally at depths of 50 to 80 feet and serve as an aquiclude to the overlying, shallow aquifer. The flood-plain alluvium in Hatch is about two miles wide, and 50 to 60 feet thick. Sands, gravels and clays deposited on top of the clay beds provide a highly permeable formation with continuity to the nearby river, as well as to local irrigation drains (King, et al, 1971)

The Santa Fe Group is composed of alluvial and basin-fill deposits with some basalt flows found in the valley. The lower limit of the Santa Fe Group is placed above the volcanic and associated sedimentary rocks of the Thurman Formation (Tertiary). The upper limit is the surface of the youngest basin-fill deposits predating the middle Pleistocene entrenchment of the Rio Grande (Wilson, et al, 1984).

Overlying the Santa Fe Formation is the post-Santa Fe group from late Pleistocene and Holocene periods. In the Hatch area, these deposits are about 50 feet thick and consist of inter-fingering lenses of sand, clay and gravel (King, et al, 1971). Samples taken from 95 shallow investigatory borings show these soils to be typically classified as A-2-4 (AASHTO). Gradations taken of selected Shelby samples show a uniformity coefficient of 1.2.

The project planned for Hatch included major reconstruction of the primary thoroughfare, Franklin Street. Drainage problems in the village were also to be addressed with the installation of storm sewers with a pumping station and forced main to alleviate street flooding. With groundwater less than six feet below ground surface, the roadway construction project was forced to deal with the additional problem of de-watering. A series of wells were planned along the corridor, to lower groundwater levels below the zone of excavation. During the preliminary planning stages, the de-watering wells were moved to the adjacent block, along Railroad Avenue. The wells were drilled 48 to 52 feet in depth, and were 32" inches in diameter, cased with 0.060 well screen. Production from these wells was from 275 to 350 gallons per minute (GPM). Transmissivity was estimated from data collected during the de-watering at 18,930 gpd/ft. Groundwater was discharged to the Colorado Drain, under permit to the Elephant Butte Irrigation District (EBID).

Construction commenced in late summer of 1990 and proceeded as anticipated until that fall, as the installation of the forced main and storm sewers approached the corner of Hall and Franklin Streets. This was to be the location of the pump station for the forced main outflow. The NMSHTD was engaged in a condemnation action against the owner of a service station located at this crucial intersection. The property owner had removed three underground storage tanks (UST's) prior to demolition of the station. Although the New Mexico Underground Storage Tank Bureau (NMUSTB) was relatively newly-formed, well-developed procedures were in place for tank removal. The NMSHTD attempted to enlist the cooperation of the New Mexico Environment Department (NMED) in certifying that the station was not impacted, however, the NMED's cooperation was limited. Although normally removed under the supervision of an employee of the NMED, the three UST's at the Hall/Franklin Street station were removed under the direction of the Village Fire Chief, who, without benefit of any instrumentation, declared the facility as un-impacted. Through Right-of-Entry granted by the court, the Department was proceeding with construction when construction workers began to notice a strong petroleum odor from the excavated material. Demolition of the former service station was underway when an additional UST was encountered. Gasoline contaminated soils were observed in the vicinity and the District Construction Engineer was notified.

With no departmental procedures established for this type of occurrence, the District Construction Engineer turned to the only professional help available, a state contractor for UST removal and testing. This company responded immediately with a fleet of equipment and began an emergency remedial action at the Hall/Franklin site. De-watering activities and all highway construction was suspended with one-half of the total forced main and storm sewer construction in-place. The town was effectively cut in half, with access to the main business district reduced to pedestrian traffic only. Until the investigation of the roadway contamination was completed, all construction activities were halted. During the regulatory scrutiny which resulted from the discovery, the EBID no longer could accept the discharged groundwater. Therefore, prior to re-initiation of construction, a method of disposal of pumped groundwater was necessary.

The NMSHTD Geology and Foundation Exploration Unit was contacted and responded to provide drilling to support the remedial contractor. Over the next six months, 95 soil borings and groundwater wells were installed around the vicinity of the Hall/Franklin Site under the direction of the state's contractor. As the investigation continued difficulties arose with the environmnental contractor and the NMSHTD Geology Unit became increasingly more involved, learning the regulatory framework for UST removal and investigation. Working with the state Environment Department's Project Manager, the investigation was completed and a re-design of the project commenced to continue the roadway project which had been stalled upon discovery of the impacted soils and groundwater.

Soils which had been excavated from the trench prior to discovery of the impacted soils at the Hall/Franklin site had been used as fill material, creating a second site which had to be addressed. The state's consultant had indicated that this area was highly impacted and required extensive remediation. Later investigation by the NMSHTD Geology Unit found that the levels of petroleum compounds in these stockpiled soils were well below the state's soil remedial action levels.

The project re-design was complicated by the need to de-water and dispose of the potentially impacted groundwater. A 13 acre retention pond, located on state land, was constructed approximately two miles outside of the project limits. A groundwater discharge permit was obtained and an extensive pre-construction investigation was initiated including soil sampling and installation of four groundwater monitoring wells. This investigation included a broad range analytical program of selected soil samples and groundwater analysis including pesticides, herbicides, metals and petroleum products. Undisturbed samples were tested utilizing a static head permeameter determining that the area was underlain with clay materials which could be compacted to 10^{-8} permeability.

De-watering wells were re-designed with the screened interval below the piezometric surface to minimize the pumping of the light, non-aqueous phase liquids (LNAPL's) and a periodic monitoring of groundwater quality from the discharge stream determined that at no time did the water exceed state groundwater standards. Approximately 32 million gallons of water were evaporated during the remaining project construction activities. Roadway construction was completed in the summer of 1991, nearly nine months beyond the scheduled completion, at an increased cost of approximately $1,000,000. Two years later litigation began between the remedial contractor and the state in which the Department settled out of court.

ESTABLISHMENT OF GUIDELINES

Subsequent to the Hatch experience, the NMSHTD and the Federal Highway Administration (FHWA) formed a committee to explore and develop procedures and guidelines to avoid a similar occurrence. The committee included members of the ROW Bureau, the Environmental Section, the Office of General Counsel, and the Geology and Foundation Exploration Unit. With the guidance of the FHWA and assistance of the Arizona Department of Transportation and other contacted agencies, the Department's Handbook of Hazardous Waste Management was developed. This document established procedures for discovery of unknown materials within the roadway, preliminary design activities and created the position of Hazardous Material Coordinator to act as liaison between the Department and environmental agencies.

The Handbook of Hazardous Waste Management was intended to serve as a framework to build upon, rather than a final document. The handbook has continued to evolve through the interim period, however, is still seen as an evolving process.

The establishment of the Hazardous Materials Coordinator (Coordinator) proved crucial to the establishment of a policy. The Coordinator currently oversees a staff of two geologists and two equipment operator/drillers conducting in-house investigations as well as providing technical oversight of state contractors.

HAZARDOUS WASTE PROCEDURES
LEGAL PERSPECTIVE

The discovery of hazardous waste sites can have tremendous impacts on project planning, budgeting and programming. The DOT must be familiar with federal and state hazardous waste laws and regulations and have up-to-date knowledge of procedures and techniques approved for hazardous waste site assessments, investigation and remediation. In the absence of this guidance, DOT staff members can expose themselves and their Department to considerable liability and risk.

The DOT's exposure, in cases where hazardous materials are present, could be substantial. On one project in Hartford, Connecticut, a former landfill was discovered on a wetland creation site. The material, while not classified hazardous, required disposal in an approved disposal site. Costs to the Connecticut DOT for disposal of the contaminated material exceeded four million dollars. In addition, the project was delayed over three months and the DOT incurred additional equipment, manpower and work acceleration costs over $750,000. The existence of contaminated material in the wetland creation site resulted in costs for environmental studies to determine if the site can now support a viable wetland. The project costs increased from $150,000 to a total expenditure exceeding five million dollars.

This case is slightly unusual but shows the exposure that highway Departments have. New Mexico is not without its problems with hazardous waste. When building highways, the

Department may encounter varied kinds of contamination, including old dumpsites, remains of creosote processing plants, UST's, and dry cleaning fluids are the most common and by far the largest problem. In general, UST's which were used to store petroleum are the largest problem. Project delays and substantial costs incurred for environmental site assessments and remediations have accompanied these projects.

The Office of General Counsel of the Highway Department has often been called in at the time of discovery of UST's during construction. A frequently asked question is what is the Department's responsibility for the clean-up? This simple question is extremely difficult to answer, depending on whether the tanks that are leaking are owned by the Department or are located outside the right-of-way.

There is a myriad of laws that may need to be complied with related to contamination discovered. Under federal law, CERCLA, SARA, and RCRA Laws may apply, depending on the status of the generator or the released contaminant. In addition to these Federal laws and regulations, the State of New Mexico also has laws and regulations related to surface water, ground water and underground storage tanks. The DOT may also be sued under a variety of common law directives including negligence, trespass and nuisance. The potential liability due to the exposure to the contractor from the contaminated materials is of substantial concern.

In terms of responsibility and liability under CERCLA, UST regulations or Common Law, the person responsible for the clean-up is generally considered to be (depending on which laws apply) one or all of several people; the owner of the property, the operator of the facility and the transporter to the site. This liability is considered as joint and several, meaning that any and all parties share the responsibility. As a "deep pockets" agency, the state DOT often finds itself responsible for these costs. Added to this role is the DOT's need to meet construction schedules and program needs, necessitating action toward project completion.

The NMSHTD's establishment of hazardous waste procedures is designed to minimize its liability and risk. These procedures were developed to minimize the possibility of encountering contamination at a time during the development of the project that will allow for a proactive approach toward avoidance or remediation of the contamination, hopefully without stopping the project thereby reducing costs. The NMSHTD's goal is to look at every phase of the project and establish procedures for best determining if there is a possibility of contamination and developing procedures for dealing with that contamination if it is found during the project.

OBJECTIVES OF A TRANSPORTATION CORRIDOR ASSESSMENT

The first goal of a transportation corridor assessment is to identify the contamination property before the property is acquired. At that time, the nature of the contamination can be assessed and the risks for continuing the project as planned can be evaluated. At this stage, a redesign of the project can be the solution. Right-of-way bureaus can become involved in determining if the landowner will clean up the property prior to acquisition. If the property is acquired, the DOT may want to include language which indemnifies the Department in case of future found contamination.

If the property cannot be avoided and it is determined to go forward with the project in spite of the risks involved, the Department will need to investigate and remediate the site. For UST's, this requires strict compliance with very detailed UST regulations. In New Mexico the Ground Water Protection Act (GWPA), provides for reimbursement for investigation and remediation of underground storage tank sites. Under GWPA, if the owner of the tanks is in "substantial compliance" with the regulations, the Department is eligible for reimbursement of expenditures over $10,000. Therefore, the Department can obtain reimbursement if it acquires the property and removes the tanks. However, if the Department doesn't take ownership of the tanks, it cannot be reimbursed and must pay for the entire cost of remediation. Although NMSHTD has not yet brought a cost recovery action against a Potentially Responsible Party, they may do so in the near future. The Illinois Department of Transportation recently completed a major construction project in which each phase of the environmental investigation and remediation was completed with an eye toward cost recovery. The resulting litigation, however, found for the property owners rather than the DOT.

Having to clean up just one major unexpected hazardous waste site can be devastating on a state highway program. Preventative measures taken in the early phases of a project and procedures for dealing with contamination when encountered is providing the Department assurances that its primary mission of building roads is not undermined by the unexpected discovery of a hazardous waste site.

Each phase of the corridor assessment is designed to provide the DOT with knowledge of site conditions which may impact project goals. The NMSHTD took a three-phased approach toward transportation corridor assessments including a Preliminary Site Assessment (PSA), an Initial Site Assessment (ISA), and a Detailed Site Investigation (DSI). Each investigatory phase provides information which allows an opportunity to modify the project scope to avoid a potentially impaired property. Taken early enough in the project planning stage, environmental corridor investigations provide a valuable tool to predict project schedules, project costs, and preferred alignments.

A determination of the regulatory compliance status of an impaired property allows the Potentially Responsible Persons (PRP's) to be established prior to acquisition. This may allow the DOT to negotiate more favorable conditions of acquisition. The DOT should be careful to avoid the appearance of impropriety during these types of investigation by utilizing the services of an outside party to establish impairment of the properties to be acquired.

CORRIDOR ASSESSMENT METHODOLOGY

Under the approach established by the NMSHTD, a methodology for corridor assessments was established which has proven successful in numerous projects. All phases of investigation are strongly field oriented, with supporting document review and data base searches. The initial phase of investigation, the Preliminary Site Assessment (PSA), is non-intrusive (no investigatory borings or drilling) and consists of a review of active and inactive installations that are, or have been engaged in the generation, dispensing, or use of any hazardous substances or materials. The DOT's goal is the identification of potential contaminant sources, rather than an quantification of risks and contaminants. Included in these investigations are literature reviews including topographic maps, geologic maps and aerial photographs, SARA Title III reports, geotechnical data, CERCLIS, RCRIS and the appropriate SRA Leaking Underground Storage Tank (LUST) lists. Specific site knowledge and experience developed through the completion of corridor assessments allows the establishment of locations to be investigated through an extensive interview process as well as a thorough document review.

On-site inspections of the roadway corridor is an important phase of the investigation, with the investigator noting subsurface geology, soil types, vegetative stress, surface staining, discharges, and other unusual conditions (discharges). The field personnel should note the potential presence of septic tanks and leach fields, and note waste handling procedures at inspected facilities. Interviews of local residences, regulators, public officials or other persons with site knowledge should be conducted, utilizing some form of an interview summary checklist. In conducting these investigations, the DOT or consultant may utilize the services of an environmental database service to research the proposed corridor for potentially impaired properties.

Following the field activities described above, the collected information is incorporated into a report format providing the necessary supporting documentation including photographs, site plans, discussions of findings, limitations and references. This report is distributed to the project team members for inclusion in the final scoping report.

The completion of the PSA phase provides an opportunity to revise project scope, modify the project design, or discard alternative alignments. Avoidance of the facility should be the preferred alternative, if possible. If the alignment is fixed, the properties which have been identified as high risk sites (those which could potentially have experienced a release to the roadway corridor) can be further tested in a more quantitative manner. If, after careful review of potential alternatives indicates that the project must proceed, subsequent ISA and DSI type investigations should follow.

These subsequent investigatory phases may include intrusive testing programs which include soil borings and sample collection of soil and groundwater with a goal of quantification of the magnitude of impacts to the roadway corridor or impaired property. A project-specific Health and Safety Plan should be prepared prior to initiating any intrusive investigatory activities. Site specific sampling and analysis plans (SAP's) are developed to identify the specific target contaminant and quantify the magnitude of the release. The Initial Site Assessment is generally conducted within the Department's existing ROW. However, it may be conducted within proposed ROW if access can be negotiated. The goal of the ISA is generally a qualitative "yes/no" type investigation to determine if a release of hazardous materials has impacted the ROW. There is no attempt to quantify the extent of contamination. Careful attention to utilities and a strict investigatory procedure is required. The investigation should be completed with potential litigation in mind. It may be most appropriate to employ outside consultants to eliminate the appearance of self-investigation.

Completion of the ISA provides an additional opportunity to modify project alignments and design. By this time in the preliminary design phase, however, alignments are frequently fixed, and further investigation of the impaired properties justified. Prior to ROW acquisition a Detailed Site Investigation should be completed to determine the extent of impacts to soil and groundwater adjacent to the facility. The goal of this project phase is quantification of extent of contamination and identification of remedial options required for project completion. The consultant should be prepared to work with the DOT's design team to establish construction and health and safety procedures, and disposal alternatives for impacted soils and groundwater.

SUMMARY

A proactive approach to hazardous materials in roadway design projects can minimize construction delays and costs associated with the presence of hazardous materials within the roadway. Established guidelines and procedures provide guidance to construction and maintenance workers, as well as design staff. Highway geologists may be the most appropriate staff members to take on these duties. Their training in hydrogeology, soils analysis and geotechnical procedures provide a basis for knowledge of fate and transport of contaminants within the roadway corridor.

There are numerous potential contaminants found in the roadway corridor, ranging from leaking underground storage tanks to hazardous materials releases from transportation accidents. A close relationship between the DOT and the state Regulatory agency provides an opportunity for cooperative efforts. Memoranda of Understandings between these agencies are an effective method of communication, benefitting all.

BIBLIOGRAPHY

Conover; "Groundwater Conditions in the Rincon and Mesilla Valleys and Adjacent Areas in New Mexico"; Geologic Survey Water Supply Paper 1230; 1953.

King, Hawley, Taylor, Wilson; "Geology and Ground-Water Rosources of Central and Western Dona Ana County, New Mexico"; Hydrologic Report 1, Water Resources Research Institute; 1971.

Seager, Hawley; "Geology of the Rincon Quadrangle, New Mexico"; New Mexico Bureau of Mines and Mineral Resources Bulletin 101.

Wilson, White, Orr, Roybal; "Water Resources of the Rincon and Mesilla Valleys and Adjacent Areas, New Mexico"; New Mexico State Engineers Technical Report 43.

"Hazardous Wastes in Highway Rights-of-Way"; National Cooperative Highway Research Program Report 351; 1993.

"Handbook of Hazardous Waste Management"; New Mexico State Highway and Transportation Department; 1992.

WASTE FOUNDRY SAND IN SUBGRADE AND CONTROLLED LOW STRENGTH MATERIAL

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Abstract: The metal casting, or foundry industry, uses sand to form molds and cores in the manufacture of castings. While much of this foundry sand is recycled through the casting process, there is inevitably some waste generated.

Promulgation of different kinds of environmental regulations, both at the federal and state level have in many cases resulted in costly land disposal facilities for these waste foundry sands. These environmental regulations, together with a current atmosphere of recession in the United States are impelling foundries to consider innovative and constructive uses for managing their wastes. This paper reports the evaluation of waste foundry sand in subgrade and in controlled low strength material.

1.0 Introduction

Waste foundry sand (WFS) is a by product of the casting industry which results from the molding and core making processes. The bulk of this WFS is non-hazardous and is currently deposited in landfills. Three different types of WFS were tested. Seven were from green sand processes. Two were from chemically bonded processes and one from the shell mold process. Samples from green sand are designated as G, chemically bonded as C and shell molding as S. A description of the different types of WFS are given by Javed (1992). The scarcity of landfill space and increase in tipping fees have stimulated the pursuit of non-landfill disposal or beneficial reuse.

A project was undertaken with the cooperation of Indiana Cast Metals Association (INCMA) to evaluate different beneficial reuses of WFS in highway construction. The different applications of WFS which include embankment, subgrade, asphalt concrete and controlled low strength material (CLSM) were evaluated. Previous work for the suitability of WFS in embankments has shown that these materials have good shear strength properties, relatively higher compressibilities and are of low permeability as compared to natural fine sands (Javed and Lovell, 1993). It was shown that when as much as 15% of WFS is replaced in conventional aggregates, the parameters of the asphalt mixture were not very different from that using conventional materials (Javed, Lovell and Wood, 1994). This paper presents the suitability of using WFS in subgrade and in CLSM.

2.0 Subgrade

The response of subgrade material under dynamic loading that simulates traffic is different from that under static loads, but due to the simplicity and popular use of static tests such as the California Bearing Ratio (CBR) test, they are often used.

2.1 California Bearing Ratio (CBR)

The CBR test is considered as a static test to evaluate and classify pavement support. The test indicates the relative resistance of material to a punching shear failure (Barksdale, 1991). The values of CBR on the selected foundry sands were determined in accordance with ASTM D 1883-87. At least three specimens were compacted in a 6 inch mold, with a spacer according to ASTM D 698-91, covering the moisture range wet and dry of optimum. The CBR of each specimen was obtained for the dry and the soaked condition. The correction of load-penetration curves were carried out following the correction procedure of ASTM D 1883-87.

Figures 1 through 4 show the moisture-unit weight-CBR relations of foundry sands. In general the 0.1-in. CBR value was less than the 0.2-in.value. A similar behavior was also reported by (Huang, 1990) and (Franco and Lee, 1987). In the case of green WFS samples, the sharp decrease in as-compacted CBR which occurred with increasing moisture content is thought to be due to fines acting as a binding agent at low moisture contents and as a lubricant at high moisture contents. The as-compacted CBR of C2 and R1 did not vary significantly with increase in moisture contents. The soaked CBR values for all the cases were generally found to vary in accordance with an increase or decrease in unit weight.

Basic criteria for the design of flexible pavement layers and the supporting subgrade are often based on the CBR values. Approximate correlations between CBR values, various soil classes and the rating of soils for subgrade, subbase or base light-traffic pavements are available. It is found that CBR values less than 5 represent a poor subgrade for light-traffic pavements, values less than 20 are unacceptable for light-traffic subbases, and values less than 80 are unacceptable for base courses. It can be seen from Figures 1 through 4, that as long as moisture content is controlled within -3 percentage points of moisture content and optimum moisture content, these WFSs are suitable as subgrade materials.

2.2 Resilient Modulus

In order to characterize WFSs for pavement response under traffic, these were also tested under dynamic stresses. The resilient modulus (M_r) is a dynamic test response defined as the ratio of repeated axial deviator stress (σ_d) to the recoverable or resilient axial strain (ε_r)

$$M_r = \frac{\sigma_d}{\varepsilon_r} \qquad (1)$$

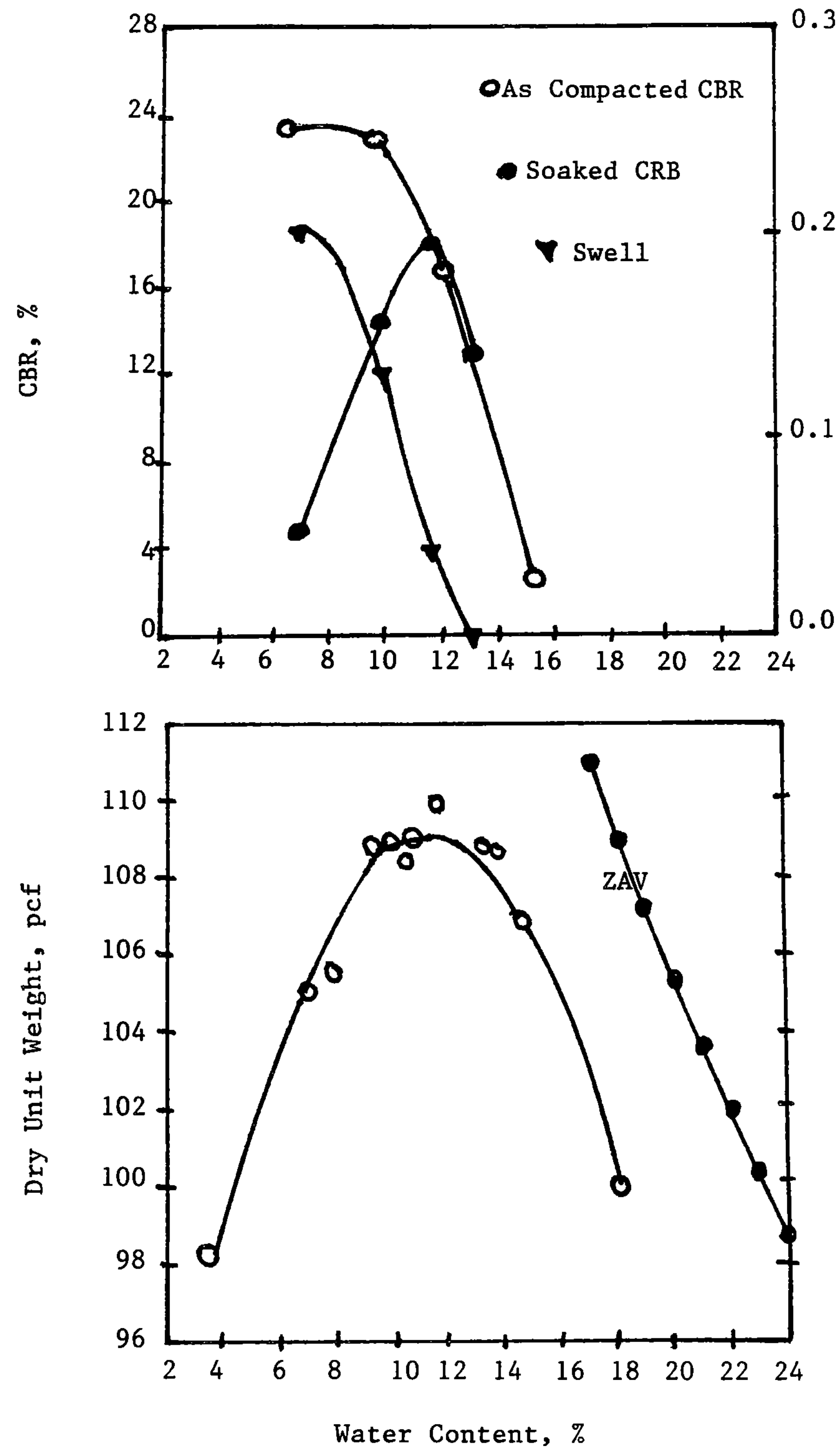

Figure 1 Moisture-unit weight-CBR relationships for G1

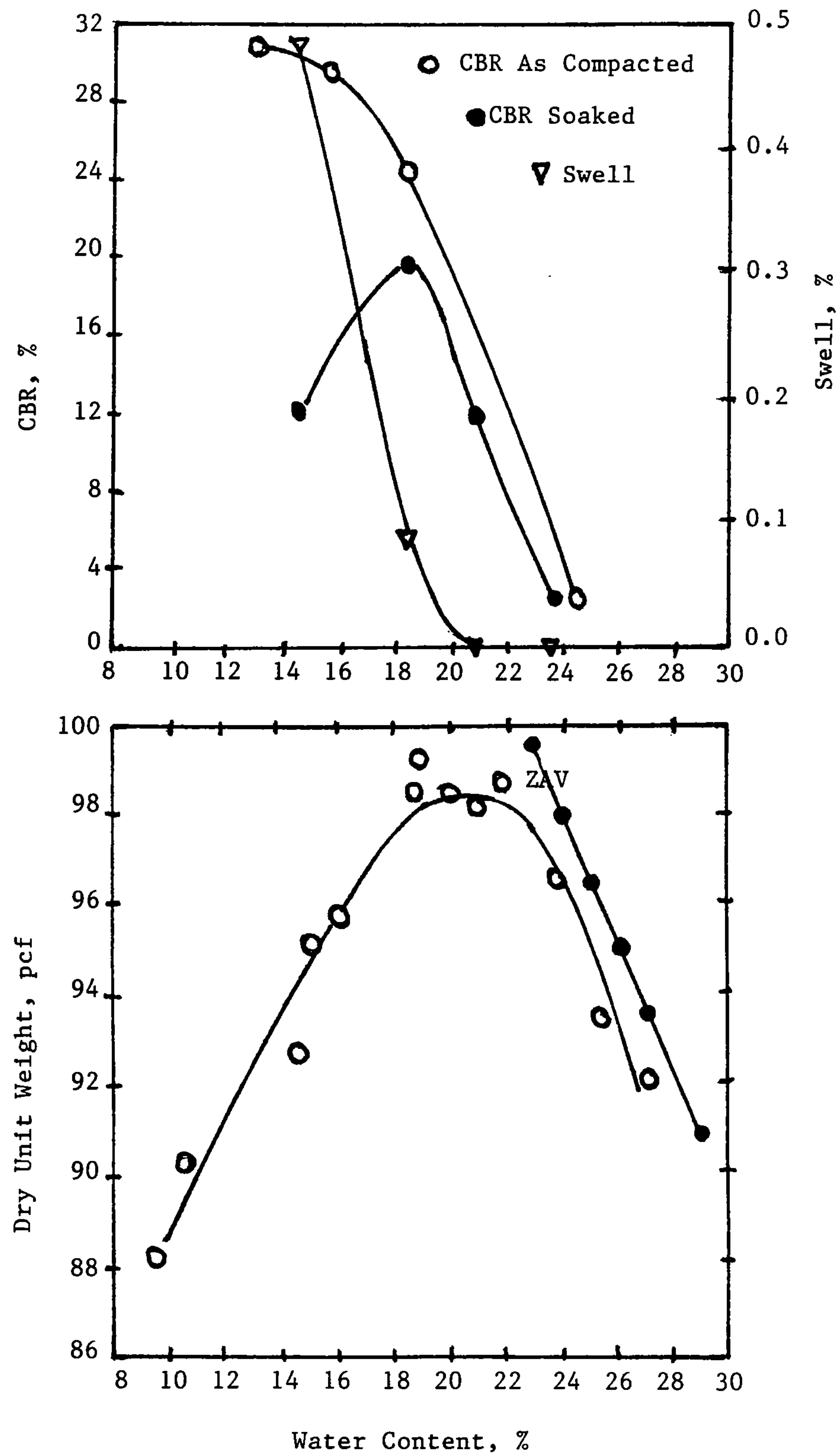

Figure 2 Moisture-unit weight-CBR relationships for G3

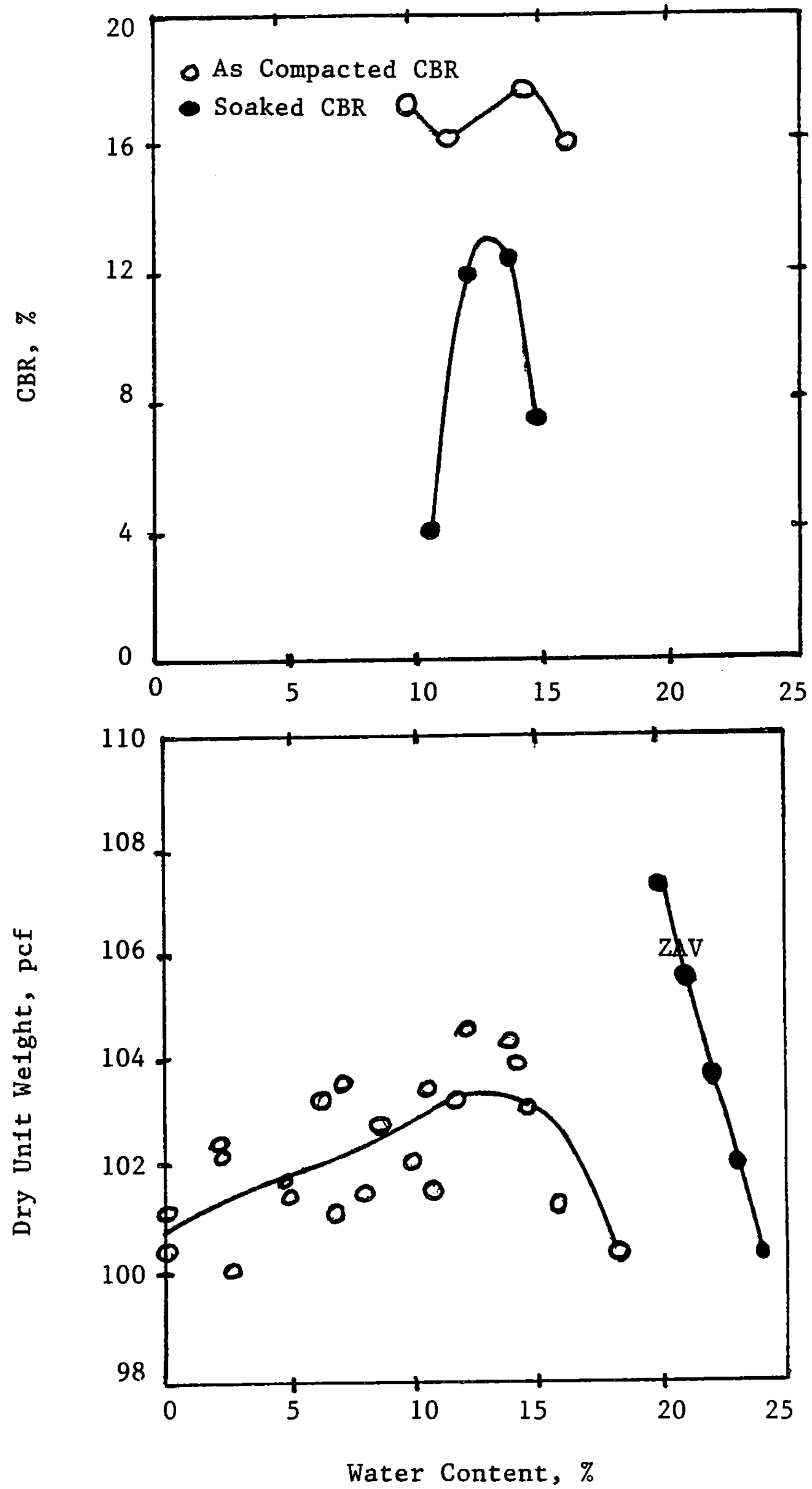

Figure 3 Moisture-unit weight-CBR relationships for C2

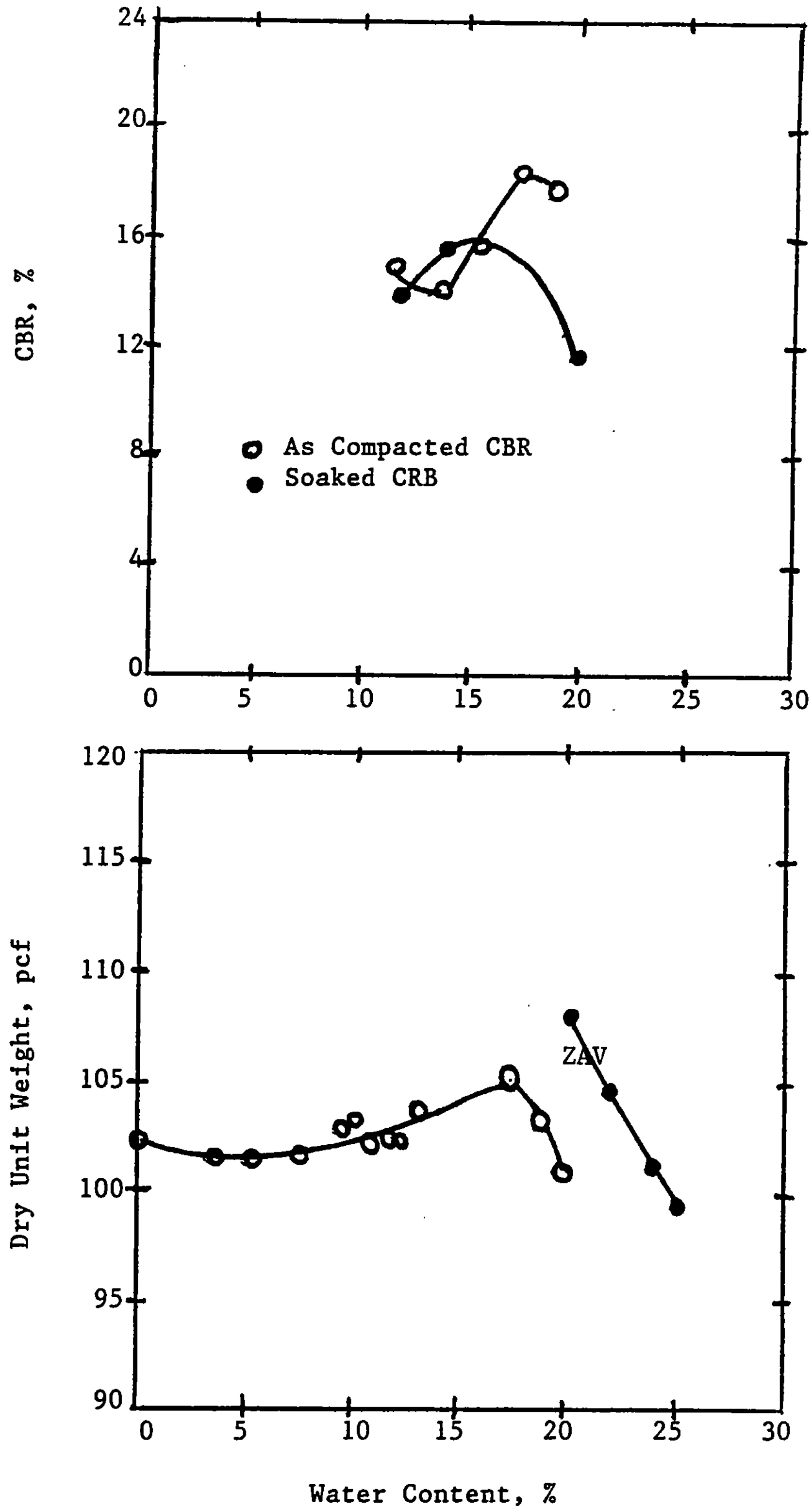

Figure 4 Moisture-unit weight-CBR relationships for R1

The testing procedures in this research conformed with AASHTO Designation T 274-82, "Standard Method of Test for Resilient Modulus of Subgrade Soils". The resilient modulus tests were conducted on G1, G3, S1 and raw sand (R1) at different moisture contents and initial unit weights. The samples G1 and G3 were prepared in a 2.8 in. diameter mold using impact compaction. A 2.8 in. mold was chosen to conserve quantities of sample required. Other reasons were the ease and speed of preparation, and the uniformity obtained. Vibratory compaction was used for samples S1 and R1 and they were prepared in a 4 in. diameter split mold. In order to determine the influence of the method of compaction on cohesionless sands, M_r tests were also performed on R1 using impact compaction in a 4 in. diameter split mold. For both impact and vibratory compaction on S1 and R1, the rubber membrane which is typically used for the preparation of granular specimens, was not used. In order to reduce the frictional force between soil and mold, the mold was covered with a plastic wrap which was later removed after the mold was split. All the samples were prepared to produce a length to diameter ratio of about 2. Samples G1, S1 and R1 were tested as granular soils. Sample G3 had about 36% fines and was therefore cohesive as per AASHTO classification (AASHTO M 145). Therefore G3 samples were tested as cohesive soils.

Previous studies have indicated that the resilient modulus (M_r) can be conveniently expressed in terms of bulk stress (θ)

$$M_r = A\theta^B \tag{2}$$

where θ is the first stress invariant ($\sigma_1 + \sigma_2 + \sigma_3$) and A and B are constants obtained from regression analysis of the test results and depend upon the type of material and physical properties of the specimen during the test.

Results of resilient modulus tests reduced by the above equation for R1 and S1 are presented in Table 1. It may be observed from Table 1 that the compaction water content and initial dry unit weight by the same method of compaction assumed no particular relationship with the A and B parameters for these uniformly graded cohesionless sands. This evidence agrees with the previous test results reported by Hicks and Monismith (1971), Smith and Nair (1973) and Lee (1993). An average value of constants A and B is therefore determined (shown in Table 2) because of their non dependency on moisture content and initial unit weight by the same compaction method.

In order to determine the influence of compaction method on M_r, the M_r values at different bulk stresses are shown in Table 2 using the average values reported in Table 1. Rada and Witczak (1981) have reported that most highway design structures result in bulk stress values near θ = 10 psi (for subbase) and θ = 20-40 psi for bases. For subgrade, the θ values between 5 to 10 psi are of particular interest. It can be observed from Table 2 that vibratory compaction resulted in a small increase in M_r values. Lee (1993) has also reported an increase in modulus values using vibratory compaction on dune sands. It may be noted that the increase in unit weight values by vibratory compaction over impact compaction was very small for R1 ($\Delta\gamma$ = 1-2 pcf), whereas it was very prominent for the dune sand reported

Table 1 Modified Resilient Modulus Test Results for R1 and S1

Test ID	W/C %	γ_d psf	A	B	R^2	Note
R11V	4.49	103.8	9481.3	0.413		Vibratory
R12V	7.93	105.2	7506.7	0.405	0.89	Vibratory
R17V	10.36	105.4	7494.8	0.435	0.81	Vibratory
R13V	12.03	103.9	8479.8	0.395	0.85	Vibratory
R14V	16.27	106.5	8096.3	0.405	0.84	Vibratory
			8212.2	0.411		Average
R11I	8.50	102.5	6409.0	0.435	0.83	Impact
R13I	12.88	102.3	7862.9	0.433	0.86	Impact
R15I	15.18	103.7	6801.3	0.446	0.86	Impact
R14I	17.10	104.9	7351.7	0.434	0.94	Impact
			7106.2	0.437		Average
S11	8.61	103.0	3875.9	0.624	0.76	Vibratory
S12	10.36	103.7	3718.1	0.562	0.94	Vibratory
S15	11.44	102.8	2873.5	0.670	0.87	Vibratory
S13	14.88	103.8	4718.0	0.549	0.78	Vibratory
S14	16.79	104.8	4260.2	0.586	0.83	Vibratory
			3889.1	0.598		Average

Table 2 Resilient Modulus Values at Different Bulk Stresses

Reference	θ = 5 psi	θ = 10 psi	θ = 50 psi	Note
R1	15912	21157	40996	Vibratory
R1	14358	19437	39272	Impact
S1	10182	15412	40349	Vibratory
Dune Sand, Lee (1993)	8145	12302	32053	Vibratory
Dune Sand, Lee (1993)	5855	8843	23040	Impact
Silty Sand Rada & Witczak (1981)	4394	6753	18318	

by Lee (1993) ($\Delta\gamma$ = 5-6 pcf). It is the authors' belief that as long as the vibratory compaction does not result in a significant increase in unit weight of cohesionless sands, M_r values are little affected by the method of compaction.

Finally Table 2 also shows that the resilient modulus values for the subgrade conditions were fairly high for the virgin foundry sands as compared to the waste shell molded sand. However, the waste shell molded sand had higher resilient modulus values than values reported for dune sand by Lee (1993) and Rada & Witczak (1981). The dune sand is typically used as subgrade in northern parts of Indiana and the sand reported by Rada & Witczak was used as sand aggregate subbase blend in a Maryland study (Rada & Witczak, 1981).

The model of equation 2 does not fit green WFSs. However, in general, an inverse relationship between the resilient modulus values and deviator stress was determined. The following equation for resilient modulus (M_r) is recommended for green WFSs:

$$M_r = E\sigma_d^F \tag{3}$$

where σ_d is the deviator stress and E and F are experimentally derived constants.

The reduced results of G1 and G3 are contained in Table 3 and 4. Table 3 also contains the reduced results of G3 subjected to a granular sequence of testing for comparison purposes. This sample of G3 was prepared near optimum.

A break-point resilient modulus was considered at σ_3 = 3 psi and σ_d = 6 psi to characterize a wide range of Indiana cohesive subgrade soils (Lee et al., 1993). In this study a confining stress of 3 psi and 1 psi were considered as representative for subgrade conditions for cohesive and granular sequences of testing. Table 5 shows the effect of water content, initial dry unit weight, percentage compaction and CBR on the resilient modulus values at different values of repeated deviator stress.

It is clearly evident from Table 5 that the increase in water content had significant effect in the case of G3. In the case of G1, dry unit weight and corresponding percentage compaction were influential for the resilient modulus values. Table 5 also shows that G33 had lower modulus values than G15 considering that both were prepared near optimum by the same method of compaction and subjected to the same sequence of testing. This suggests that increasing fines in green WFS would result in lower modulus values near optimum. However, on the dry side of optimum, modulus values increase with increasing amount of fines. Therefore, in general, for the case of green WFSs, it would be advantageous to compact them on the dry side of optimum (preferably within -3 percentage points of moisture content to optimum moisture content).

Lee (1993) reported M_r on a wide variety of cohesive soils typically used as subgrade in Indiana. Most of the soils had M_r values between 4000 and 18000 psi. Finally, from Table 5, it is observed that the laboratory compacted green WFSs had comparable or higher values than most subgrade soils used in Indiana.

Table 3 Constants E and F Parameters for G1

Test ID	W/C %	γ_d pcf	σ_3 = 20 psi			σ_3 = 20 psi		
			E	F	R^2	E	F	R^2
G11	6.93	105.0	41690	-0.05	0.66	42267	-0.11	0.99
G17	7.54	106.7	40910	-0.04	0.68	39491	-0.06	0.87
G12	9.41	108.3	45045	-0.04	0.99	60916	-0.20	0.96
G15	11.39	109.1	89699	-0.19	0.98	78180	-0.19	0.95
G13	13.44	99.0	47936	-0.05	0.74	69194	-0.17	0.96
G33	20.76	98.1	41397	-0.28	0.96	42403	-0.32	0.99

Test ID	σ_3 = 10 psi			σ_3 = 5 psi			σ_3 = 1 psi		
	E	F	R^2	E	F	R^2	E	F	R^2
G11	42560	-0.20	0.92	29923	-0.19	0.80	21199	-0.70	0.85
G17	45612	-0.20	0.73	32160	-0.17	0.85	34765	-0.55	0.89
G12	66304	-0.31	0.84	53597	-0.32	0.85	47897	-0.51	0.80
G15	84410	-0.28	0.99	63364	-0.28	0.99	46702	-0.40	0.98
G13	56587	-0.23	0.87	59481	-0.36	0.86	39219	-0.47	1.00
G33	40167	-0.36	0.78	51856	-0.60	0.93	46089	-0.77	0.97

Table 4 Constants E and F for G3

Test ID	Water Content %	Unit Weight pcf	$\sigma_3 = 6$ psi			$\sigma_3 = 3$ psi			$\sigma_3 = 0$ psi		
			E	F	R^2	E	F	R^2	E	F	R^2
G3A4	13.72	93.1	48540	-0.37	0.93	47552	-0.40	0.80	48121	-0.53	0.93
G3A1	15.63	95.0	36465	-0.28	0.81	36459	-0.31	0.72	24751	-0.48	0.87
G3A2	17.63	96.8	45268	-0.41	0.99	29350	-0.24	0.81	31738	-0.39	0.98
G3A3	20.14	98.4	39753	-0.44	0.93	34534	-0.44	0.98	25731	-0.53	0.99

Table 5 Effect of Water Content, Dry Unit Weight, Percentage Compaction and CBR on the M_r Response for Green WFSs

Test ID	Water Content %	Dry Density pcf	Compaction %	CBR %	Resilient Modulus (psi)			σ_3 psi
					$\sigma_d = 3$ psi	$\sigma_d = 4$ psi	$\sigma_d = 9$ psi	
G3A4	13.72	93.1	94.6	30.7	30642	27311	19746	3
G3A1	15.63	95.0	96.5	27.5	25936	23723	18460	3
G3A2	17.63	96.8	98.4	25.3	22548	21043	17322	3
G3A3	20.14	98.4	100.0	18.2	21297	18765	13133	3
G33	20.76	98.1	99.7	16.0	19779	15849	8488	1
G11	6.93	105.0	96.2	23.4	9825	8033	4554	1
G17	7.54	106.7	97.7	23.1	18999	16218	10383	1
G12	9.41	108.3	99.2	23.1	27291	23553	15550	1
G15	11.39	109.1	99.9	18.8	29128	25888	18565	1
G13	13.44	108.1	99.0	10.6	23325	20357	13872	1

3.0 Controlled Low Strength Material

Controlled Low Strength Material (CLSM) is used for backfilling trenches, retaining walls, bridge abutments, underground tanks, culverts and utility cuts from sewer, gas, water and electric repair; as fill for abandoned tanks, wells, sewers, and manholes; and in pipe bedding. Strengths after 28 days range from 50 to 100 psi, which is higher than most compacted natural soils, but low enough to allow future excavations. Additionally, this type of material does not require compaction as does soil backfill. The latter adds expense.

The study at Purdue University pursued this application of WFS in CLSM. Five different types of mixes were prepared with the objective of proposing a most economical mix and satisfying the requirements of flowability, compressive strength and set time. Final mix proportions along with the spread, unit weight, and compressive strength data are given in Table 6. This Table also includes data, using conventional materials, from two sources. The data for Mix ES-1 is taken from Nantung (1993) while the data for Mix 1A is taken from Amon (1990). These data are included for comparison purposes. It was found that the unit weights of the CLSM mixes using WFS were lower than the conventional mixes. As the wet unit weight affects the pressures applied by the CLSM on pipes or retaining walls, mixes using WFS are better than conventional mixes in this regard. In addition, the rate of strength gain for mixes using WFS was found to be lower than for mixes using conventional materials. This fact is an advantage for long term strength development of CLSM, if later removeability is taken into consideration.

Table 6 Spread, Unit Weight and Compressive Strength for Different Mixes

Mix No.	1	2	3	4	5	1A[1]	ES-1[2]
Cement (pcy)	131	124	99	55	76	80	63
Fly ash (pcy)	436	414	158	183	139	420	321
Water (pcy)	698	745	741	738	765	561	494
WFS or Sand (pcy)	1922	1822	2134	2128	2126	2535	2774
Spread (in.)	7.5	9.0	8.2	8.1	8.2	-	5-6
Unit weight (pcf)	118	115	116	115	115	-	134
Compressive strength (psi)							
3 - Day	43	41	36	8.5	24	-	20
7 - Day	67	54	44	9	31	45	30
28 - Day	103	80	70	8	55	120	80

[1] From Amon (1990)
[2] From Nantung (1993)

Controlled low strength material should be highly flowable to allow ease of construction. It should easily flow into inaccessible spots, flowing under and around pipe to provide perfect bedding (Kepler 1986). The flowability is determined by filling an open ended 3 in. diameter by 6 inch high plastic container; slowly lifting the cylinder and letting the CLSM spread laterally on the flat surface; and measuring the spread. For proper flowability, the spread should be at least 8 inches (ACI Committee 229). The spread of the different mixes ranged from 7.5 to 9.0 inches.

To be economically feasible, CLSM should set up between 4 and 24 hours. If it sets up too quickly, it would set up in the mixing/transportation devices. If it sets up too slowly, it would delay construction too much (Kepler, 1986). For the first mix, the set times was determined based on a penetration number, i.e., penetration resistance of 400 psi. Thus the set time for this mix was at 16 hours. However, it is the authors' belief that this mixture had a set time at 3 to 5 hours based on Nantung's definition of hardening stage. For the latter mixes, the set time was determined by determining the hardening stage as proposed by Nantung (1993). Hardening stage was indicated by a penetration resistance number of 40 psi and a sudden increase in the slope of the curve of the penetration resistance. These indications guaranteed that the mix already had a load carrying capacity and had passed the accelerated period in cement hydration. Therefore proper cement hydration had already occurred.

The set time of Mix 2 was at 6 or 7 hours. Mix 4 did not set at all. Mix 5 set at between 7 and 8 hours. Generally, set time was essentially dependent on water to cement ratio (i.e. lesser the water to cement ratio, earlier the hardening stage). Comparing mixes 3, 4 and 5, this conclusion seems true. However, comparing mixes 2 and 3, this conclusion does not hold. For this case, it was the increase in unit weight of mix 3 which caused it to be more resistant than Mix 2. Thus for mixes having identical unit weights, the hardening stage is dependent on water to cement ratio.

4.0 Conclusions

1. The as-compacted CBR for green WFS decreased significantly with increasing moisture content. However, little variation with moisture content (in the as-compacted CBR values) was found for cohesionless soils, like C2 and R1.

2. The soaked CBR values for all cases were found to vary in accordance with the dry unit weight.

3. The CBR values suggest that these sands can be used for subgrade materials. However, their use as subbase and base is unacceptable.

4. Resilient modulus of uniformly graded cohesionless sands like R1 and S1 were found to be independent of the increase in compaction water content and initial dry unit weight obtained by the same method of compaction.

5. Vibratory compaction results in an increase in resilient modulus for cohesionless

sands.

6. For green WFSs, as long as the fines are low (about 6% passing #200 sieve), M_r is affected by the initial dry unit weight obtained by the same method of compaction. When the fines are more (about 36% passing #200 sieve), M_r tends to decrease with increasing water content.

7. In general, WFSs have M_r values comparable or higher than most soils used as subgrade in Indiana.

8. Wet unit weight affects the pressures applied by the CLSM on pipes or retaining walls. Thus mixes using WFS are better than conventional mixes in this connection.

9. The rate of strength gain for mixes using WFS is lower than for mixes using conventional materials. This fact is an advantage for long term strength development of CLSM, if later removeability is taken into consideration.

10. The CLSM strength was primarily dependent on the cement content. Higher cement content resulted in higher strength.

11. In general, the hardening stage of the CLSM mixes were dependent on water to cement ratio. The hardening stage was reached earlier for mixes having lower water to cement ratio. But for mixes having unlike unit weights, the hardening stage was reached earlier for mixes having higher unit weights.

12. Based on a critical review of the mixes prepared in this study, Mix 3 is best.

13. Controlled low strength materials using WFS are economical as compared to conventional mixes and their use should be encouraged.

References

1. ACI Committee 229, "Controlled Low Strength Material", Draft Report, American Concrete Institute, Detroit, Michigan.

2. Amon, J. A. (1990), "Controlled Low-Strength Material", Construction Specifier, Vol. 43, No. 12, December, pp. 98-101.

3. Asphalt Institute (1970), "Soils Manual for Design of Asphalt Pavement Structures", The Asphalt Institute, College Park, Maryland.

4. Barksdale, R. D. (1991), The Aggregate Handbook, National Stone Association, Washington, D.C., pp. 1-2 to 16-37.

5. Franco, C. A. and Lee, K. W., "An Improved California Bearing Ratio Test Procedure", Transportation Research Record 1119, 1987, Pennsylvania, pp. 91-97.

6. Hicks, R. G. and Monismith, C. L. (1971), "Factors Influencing the Resilient Response of Granular Materials", Highway Research Record 345, pp. 15-31.

7. Huang, W. (1990), "The Use of Bottom Ash in Highway Embankments, Subgrades and Subbases", Report No.: FHWA/IN/JHRP-90/4, School of Civil Engineering, Purdue University, West Lafayette, Indiana, 317 pp.

8. Javed, S. (1992), "Use of Waste Foundry Sand in Highway Construction", Interim Report, Report JHRP-92-12, School of Civil Engineering, Purdue University, West Lafayette, Indiana, May, 28 pp.

9. Javed, S. and Lovell, C. W. (1993), "Use of Waste Foundry Sand in Highway Construction", Proceedings of 44th Highway Geology Symposium, Tampa, Florida, May, 16 pp.

10. Javed, S., Lovell, C. W. and Wood, L.E . (1994), "Waste Foundry Sand in Asphalt Concrete", Preprint for 1994 TRB Meeting, Washington, D.C., 22 pp.

11. Javed, S. (1994), "Use of Waste Foundry Sand in Highway Construction", Ph.D. Thesis, School of Civil Engineering, Purdue University, West Lafayette, Indiana, May, 274 pp.

12. Kepler, W. F. (1986), "Studies on Controlled Low Strength Mortar", JHRP-86/11, School of Civil Engineering, Purdue University, West Lafayette, Indiana, 44 pp.

13. Lee, W. (1993), "Evaluation of In-Service of Subgrade Resilient Modulus with Considerations of Seasonal Effects", Ph.D. Thesis, Purdue University, West Lafayette, Indiana, 272 pp.

14. Lee, W., Bohra, N. C., Altschaeffl, A. G., and White, T. D. (1993), "Subgrade Resilient Modulus for Pavement Design and Evaluation", JHRP 92-23, Purdue University, West Lafayette, Indiana, 349 pp.

15. Nantung, T. (1993), "Design Criteria of Controlled Low Strength Materials", Ph.D. Thesis, Purdue University, West Lafayette, Indiana, August.

16. Rada, G. and Witczak, M. W. (1981), "Comprehensive Evaluation of Laboratory Resilient Moduli Results for Granular Material", Transportation Research Record 810, pp. 23-33.

17. Smith, W. S. and Nair, K. (1973), "Development of Procedures for Characterization of Untreated Granular Base Course and Asphalt-Treated Base Course Materials", U.S. Department of Transportation, FHA, Report No. FHWA-RD-74-61, 204 pp.

"Investigation, design and construction of the Spirit Lake Memorial Highway"

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ABSTRACT: The geologic environment near Mount St. Helens has been shaped by active volcanism, alpine glaciation, fluvial processes and mass wasting ranging from downslope colluvial action to large scale land sliding. These processes have resulted in the deposition of a wide variety of surficial deposits with highly variable engineering properties. Bedrock in the area consists primarily of pyroclastic and flow volcanic rocks which, in some areas, have experienced significant hydrothermal alteration. In the extreme, this has resulted in a rockmass with engineering properties closer to soil than to rock. A complete spectrum of rock quality exists between this extreme and competent, relatively unfractured rock.

The Washington State Department of Transportation (WSDOT) recently completed the first 25 mile phase of construction of State Route 504, designated the Spirit Lake Memorial Highway, located west of the Mount St. Helens National Volcanic Monument. A further 6.6 mile section into the monument itself is currently under construction. The steep topography of the Cascade Mountain Range combined with the highly variable surficial and bedrock geology presented particular challenges to WSDOT in terms of geotechnical exploration, design and construction. Project elements on the first 25 mile section of highway included ten major bridges, over 4000 feet of retaining walls, and embankments and cut slopes up to 120 feet in height. The 6.6 mile section of highway, presently under construction, includes an additional five bridges, and embankment and cut slopes up to 100 feet in height as well as stabilization measures for a large landslide.

WSDOT, along with their geotechnical consultants, undertook a detailed geotechnical exploration program to characterize the alignment for final design. The geotechnical explorations included 772 test holes totaling 36402 linear feet, 275 test pits, and 18,704 feet of seismic refraction line. Specialized in-situ electronic cone penetrometer, and self boring pressuremeter testing was also utilized to characterize shear strength parameters of sensitive soil deposits for embankment and slope stability analysis. This comprehensive geotechnical exploration and design strategy enabled many cost saving measures to be realized. These included more efficient design elements, steeper cut slopes and embankments, and maximized utilization of roadway excavation along the highway alignment. This geotechnical exploration and design strategy also minimized construction claims, and provided reliable data with which to defend the claims that did arise.

INVESTIGATION AND STABILIZATION OF A DEVELOPING LANDSLIDE HIGHWAY 28, SOUTH OF LANDER, WYOMING

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ABSTRACT

The reconstruction and subsequent realignment of Highway 28, south of Lander Wyoming, was conducted in 1985 and required traversing existing landslide deposits and the placement of 40 feet of fill over an existing spring. The spring is a State water right and the protection of this water source was considered imperative. An underdrain system was constructed to protect the flow (approximately 300 GPM) and allow the spring to outlet at the toe of the embankment.

Since August 1987, an active landslide has developed around the spring which posed a direct threat to the integrity of the spring and roadway. The slide encompasses an estimated 350,000 cubic yards and is affecting approximately 800 feet of roadway. The highway has required patching with 2,000+ tons of asphalt to keep it up to grade since movement first started to occur. Inclinometers installed in August 1987, and August 1991, indicate slide movement at a depth of 81 feet, 40 feet below the original spring outlet on natural ground. Due to accelerating movement, an emergency contract was let in the fall of 1992 to construct three toe trenches to protect drainage of the spring and act as a toe berm to stabilize the slide (roadway). The slide movement around the spring was successfully stabilized.

Movement has now started to occur immediately east of the repaired slide area, adjacent to the spring. An investigation of the additional slide movement involved installing a new slope inclinometer, field reconnaissance and aerial mapping. A landslide analysis was calculated using drill hole, inclinometer data, and surface feature mapping information. Various corrective solutions were analyzed, utilizing the XSTABL program, in order to evaluate the design which would have the greatest impact in stabilizing the slide. These designs included: 1) a shift in the alignment; 2) lowering the grade; 3) replacing the soil embankment with lightweight fill; 4) building additional toe berms. Combinations of these designs were chosen to obtain the greatest factor of safety.

The remedial action will consist of a maximum 76 foot shift of centerline, construction of a lightweight embankment using shredded tires, additional toe berms in the area of the spring, and the removal of the existing embankment in order to reduce driving forces. Removal of existing embankment will also allow for the spring to be uncovered and return to an uninhibited state similar to the way it was before the road was built. Construction is planned for the fall of 1994.

James M. Dahill

INTRODUCTION

The Double Nickel landslide is situated approximately 20 miles south of Lander Wyoming, along Highway 28 at M.P. 55.5 (see figure 1). State Highway Route 28, better known as South Pass, is a primary highway that carries traffic over the Wind River Mountains in central Wyoming. South Pass was a major feature of the Oregon Trail during the mid-19th century, and now is used as a major trucking route for hauling trona, which is mined around the Green River area and trucked over the mountains to the Burlington Northern Railroad tracks at Bonneville, located in the center of the state.

In 1985 seven miles of Highway 28 were realigned to improve the vertical and horizontal alignment and upgrade it to current Federal Highway design standards and specifications. The shift in the alignment at M.P. 55.5 required placing the roadway directly over a naturally occuring spring. The spring was identified by the State Archeologist as an old Indian campground by the many artifacts found at the site, and its water flow was registered as a State water right. The first signs of a developing landslide were noticed in 1987 around the spring, damaging the roadway and posing a direct threat to the integrity of the spring. The investigation and stablization of the roadway is the focus of this report.

GEOLOGIC HISTORY

The South Pass area is located at the southern end of the Wind River Range. State Highway 28 runs south from Lander, rising to the foothills of the Wind River Range and crossing the Contential Divide at elevation 7550 before continuing another 40 miles southwest to Farson. The range was uplifted during the Late Cretaceous-Early Tertiary Laramide orogeny and thrust to the west and south along moderately dipping thrust faults. Paleozoic and Mesozoic rocks on the eastern flank of the range dip northeast into the Wind River Basin and disappear under the Tertiary and Quaternary sedimentary cover.

The road alignment traverses Quaternary alluvium and landslide debris in the area of M.P. 55.5. The Tertiary White River Formation is present over most of the project and consists of tuffaceous siltstone, sandstone, and conglomerate with some bentonitic zones. Outcrops occurring in the vicinity of the slide are Pennsylvanian age Ten Sleep Sandstone and Amsden Formation, and the Mississippian Madison Limestone (Geological Survey of Wyoming Map Series 38, Miners Delight Quadrangle).

The alluvial deposits consist of an unconsolidated mixture of clay, silt, sand, and gravel. The landslide deposits consist of a heterogeneous mixture of gravelly clays, silts, and sands.

The Tensleep Sandstone is a thick, cross-bedded sandstone which outcrops east of the slide. The Madison Limestone is a massive limestone and outcrops to the west of the slide. These bedrock formations dip to the northeast at 12 degrees.

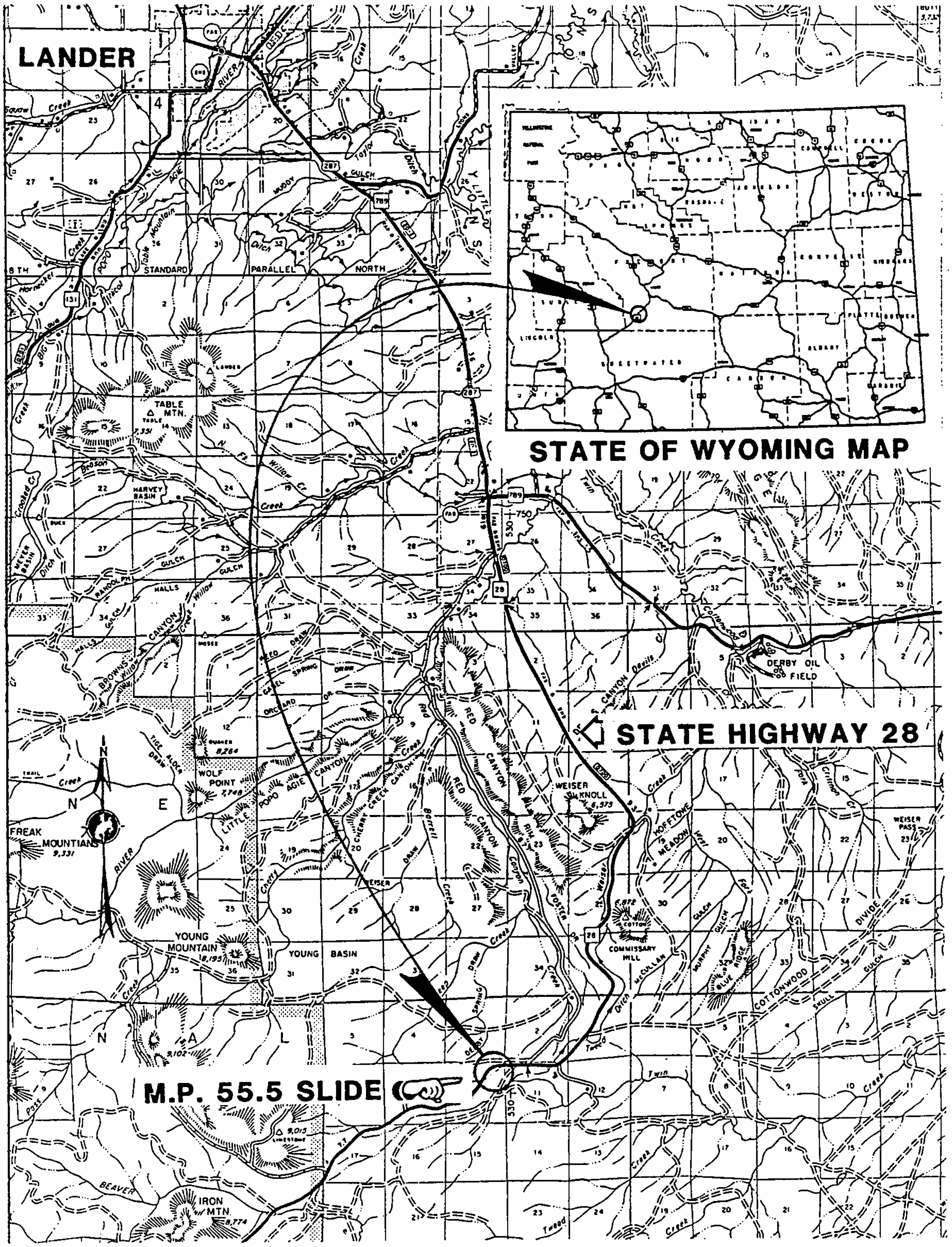

Figure 1. Location map with State of Wyoming map inset.

BACKGROUND

In November of 1981, the Geology Program conducted a Geologic Soils Profile Investigation for the new alignment. The Soils Profile report included recommendations for the construction of a detailed underdrain system to allow for unrestricted flow of the spring located at M.P. 55.5 (see figure 2). Plans were issued in June 1984, and construction work started in February 1985.

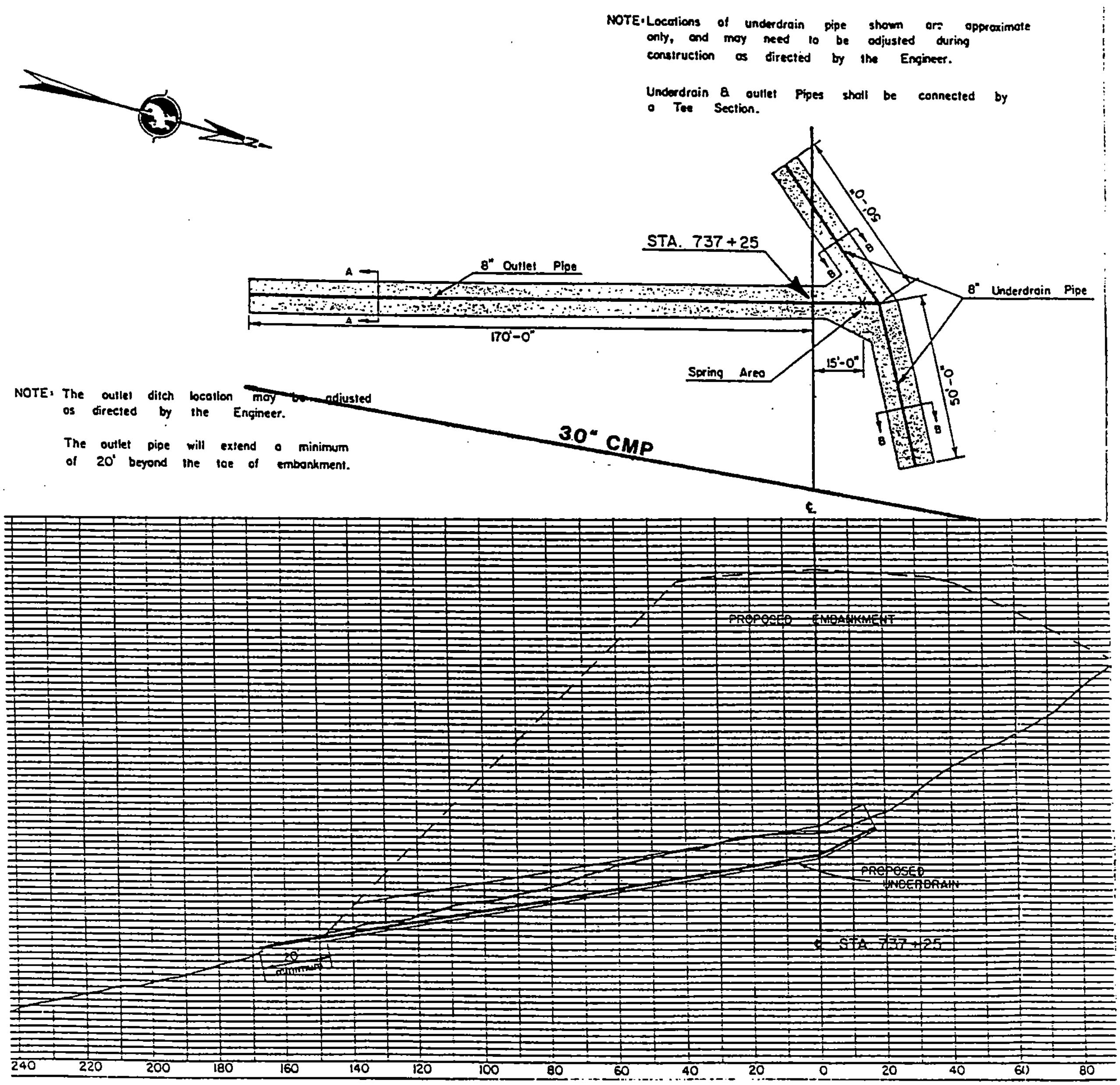

Figure 2. Diagram of the underdrain detail included in the 1985 construction plans.

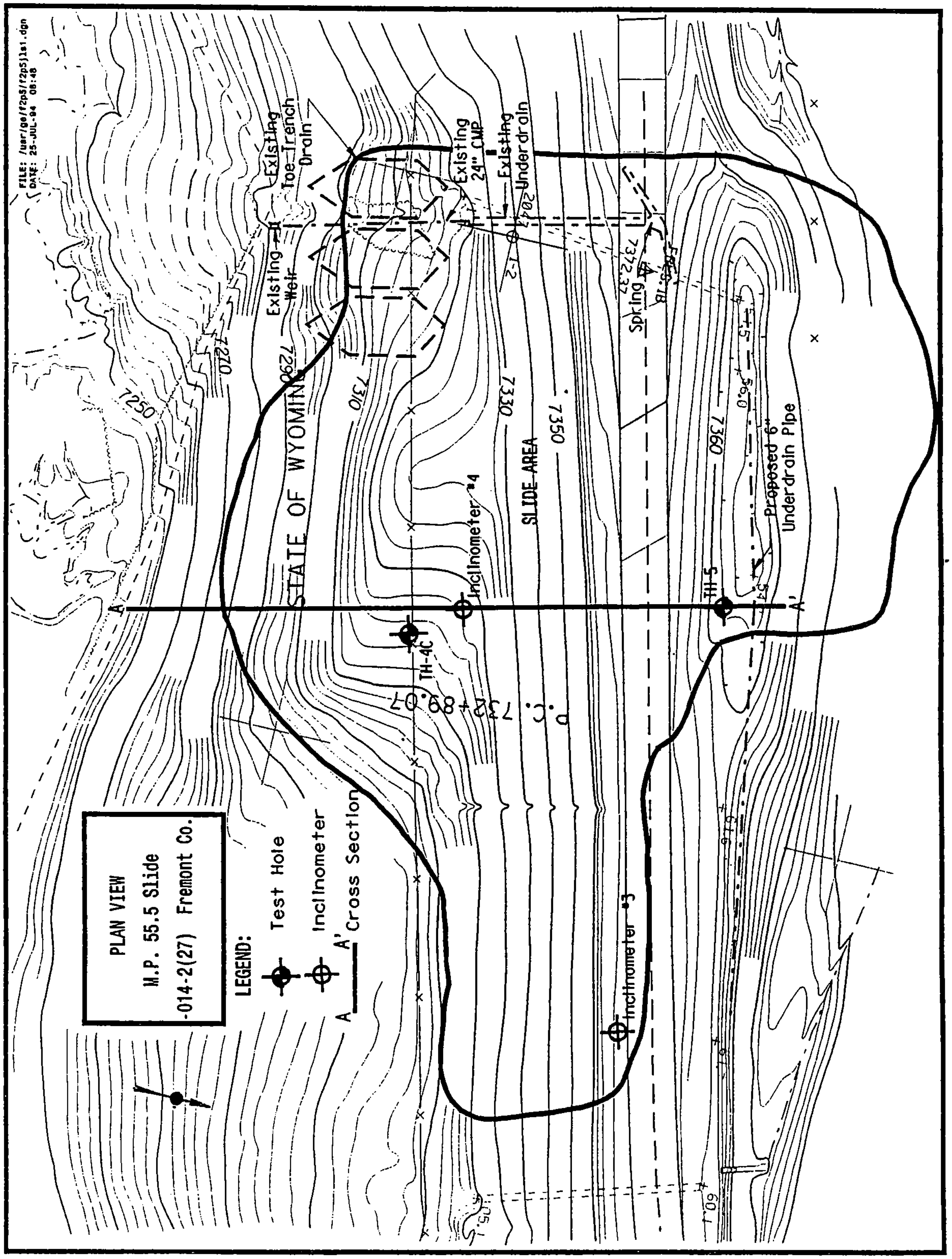

Figure 5. Plan view showing outline of slide area.

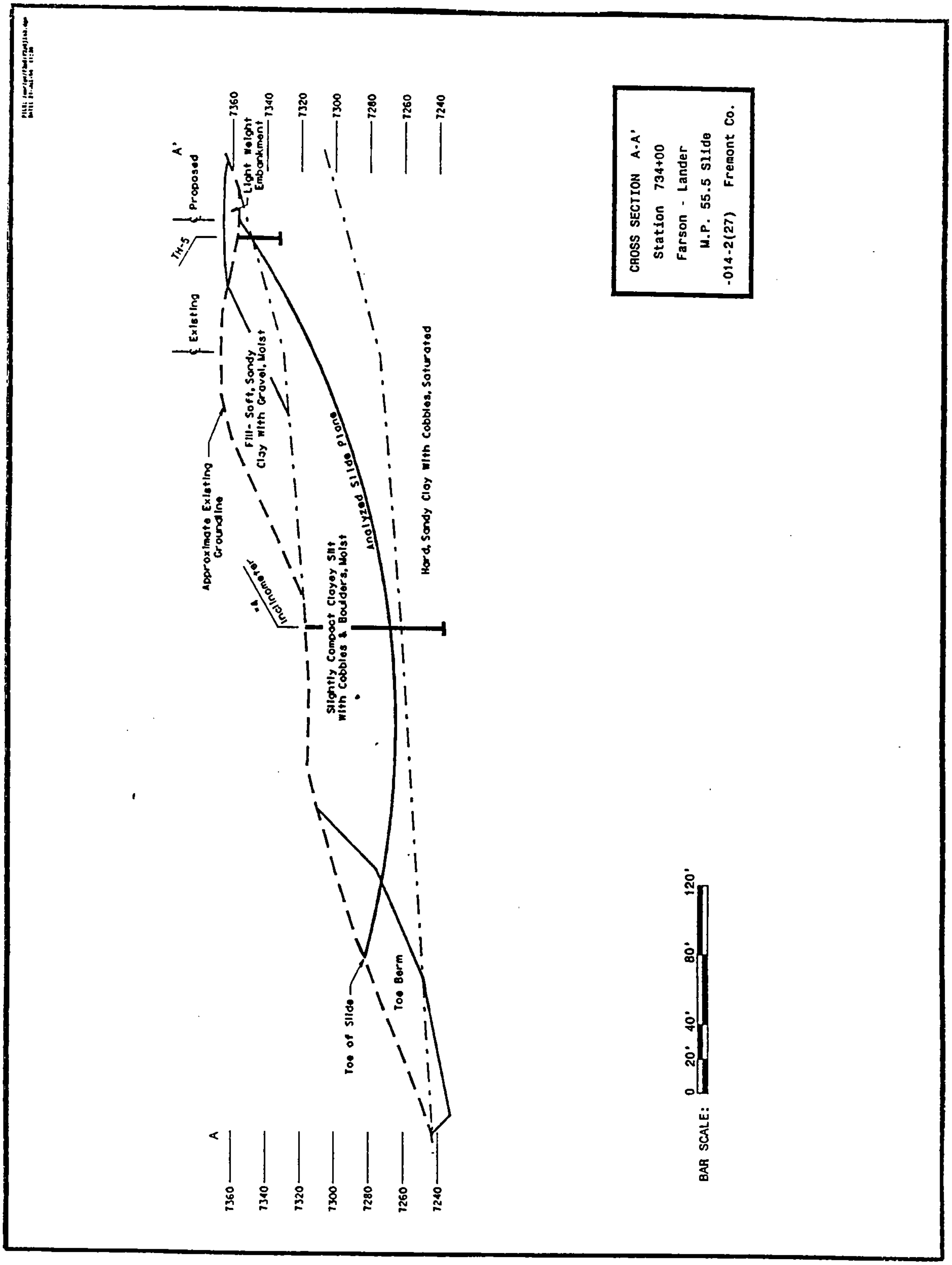

Figure 7. Cross section developed from drill hole data with analyzed slide plane.

The 40 foot high fill at the head of the slide is providing large driving forces, and any solution requires the reduction of this weight. Consideration was also given to designs that would allow for the spring to be unearthed to permit unrestricted flow. The four following alternatives were considered in the stabilization of the slide.

1) Shift in alignment: A shift of the alignment to the south kept the roadway within the active slide area and was ruled out. An alignment shift to the north required moving towards the top of the slide (increasing driving forces), however a section of the roadway at NE end of the slide could be located beyond the slide scarp. This shift configuration permitted the removal of some of the existing embankment, decreasing driving forces. The shift to the north also created the possibility of uncovering the spring at the SW end of the slide.

2) Lowering the grade: Lowering the grade 10 feet or more created only a slight increase in the safety factor. The large area effected by lowering the grade generated a large quantity of waste material and infringed on sensitive landslide areas immediately to the west of the area. Therefore, lowering the grade was ruled out.

3) Lightweight fill: Three possibilities of lightweight material were considered to replace the existing soils and reduce the driving forces.

a) Woodchips - Woodchips were used on a slide approximately one-half mile up the road and worked successfully. The unit weight of woodchips is approximately 35 pounds per cubic foot versus the existing 125 pounds per cubic foot fill. Availability and haul expense are the controlling factors. The cost of woodchips varied from $8-15 per cubic yard.

b) Shredded Tires - Shredded tires have been used in other states successfully and are about the same weight as woodchips (35 PCF). The use of tires may also help the state meet its tire disposal requirements for ISTEA. The cost of tires was $4 per cubic yard.

c) GeoFoam - Expanded polystyrene (EPS) was used in Colorado for landslide remediation. EPS has a unit weight of 1.5 PCF reducing driving forces significantly. EPS was ruled out due to its expense at $30-50 per cubic yard. (Siel, 1989)

4) Building additional toe berms: The toe berms installed in the previous contract have worked effectively. The U.S. Steel Pit located at the top of the mountain pass offers an abundant source of heavy iron ore rock (reject material) to be used in the toe berms (resisting force).

The final recommended design will consist of a combination of these alternatives (see figure 8). The existing centerline will be shifted to the north with the new fill constructed of shredded tires. The existing embankment will be removed within the limits of the realignment to create the new fill slopes. Four toe trenches will be constructed at the NE end of the slide area to increase the resisting forces (see figure 9).

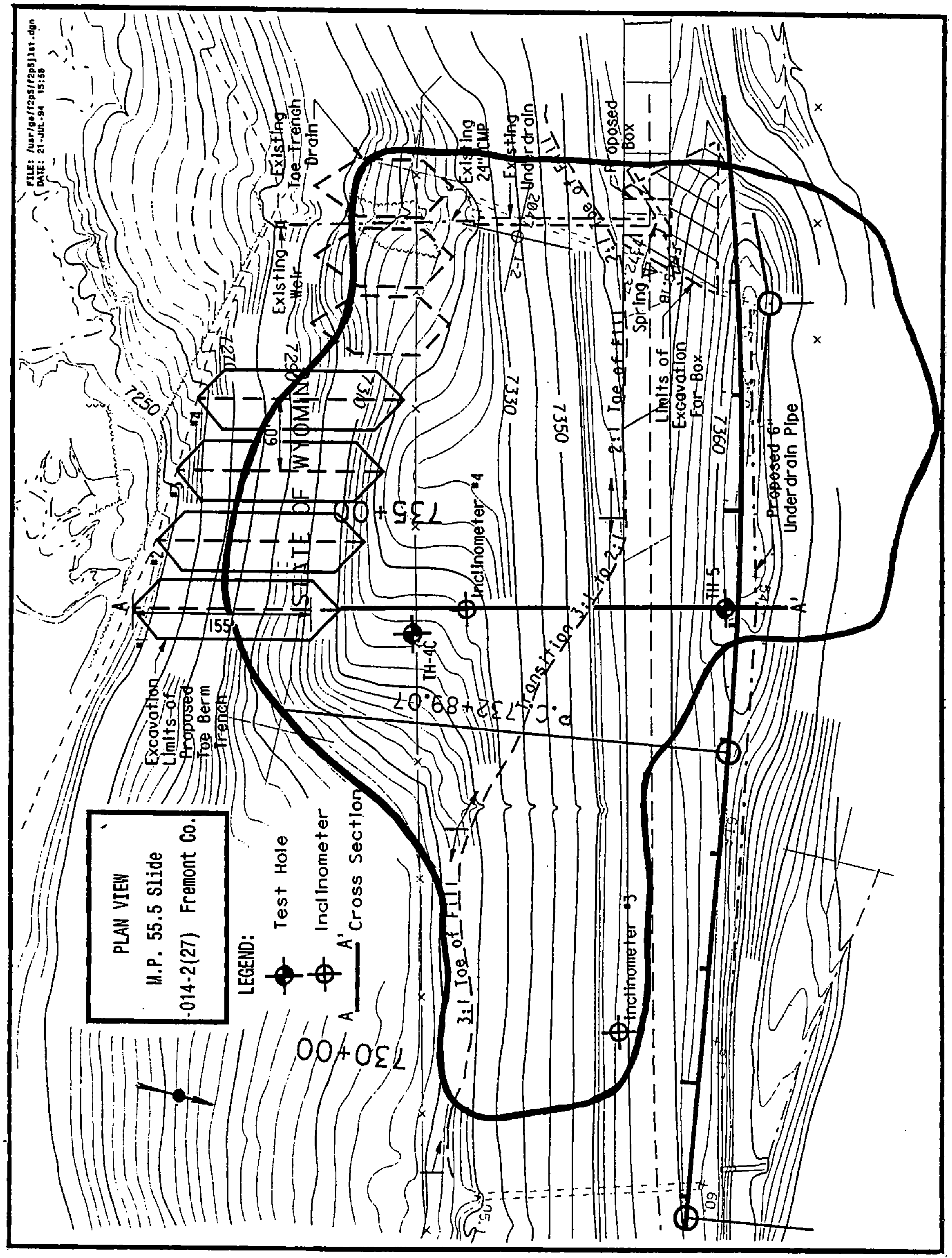

Figure 8. Plan view showing planned design recommendations.

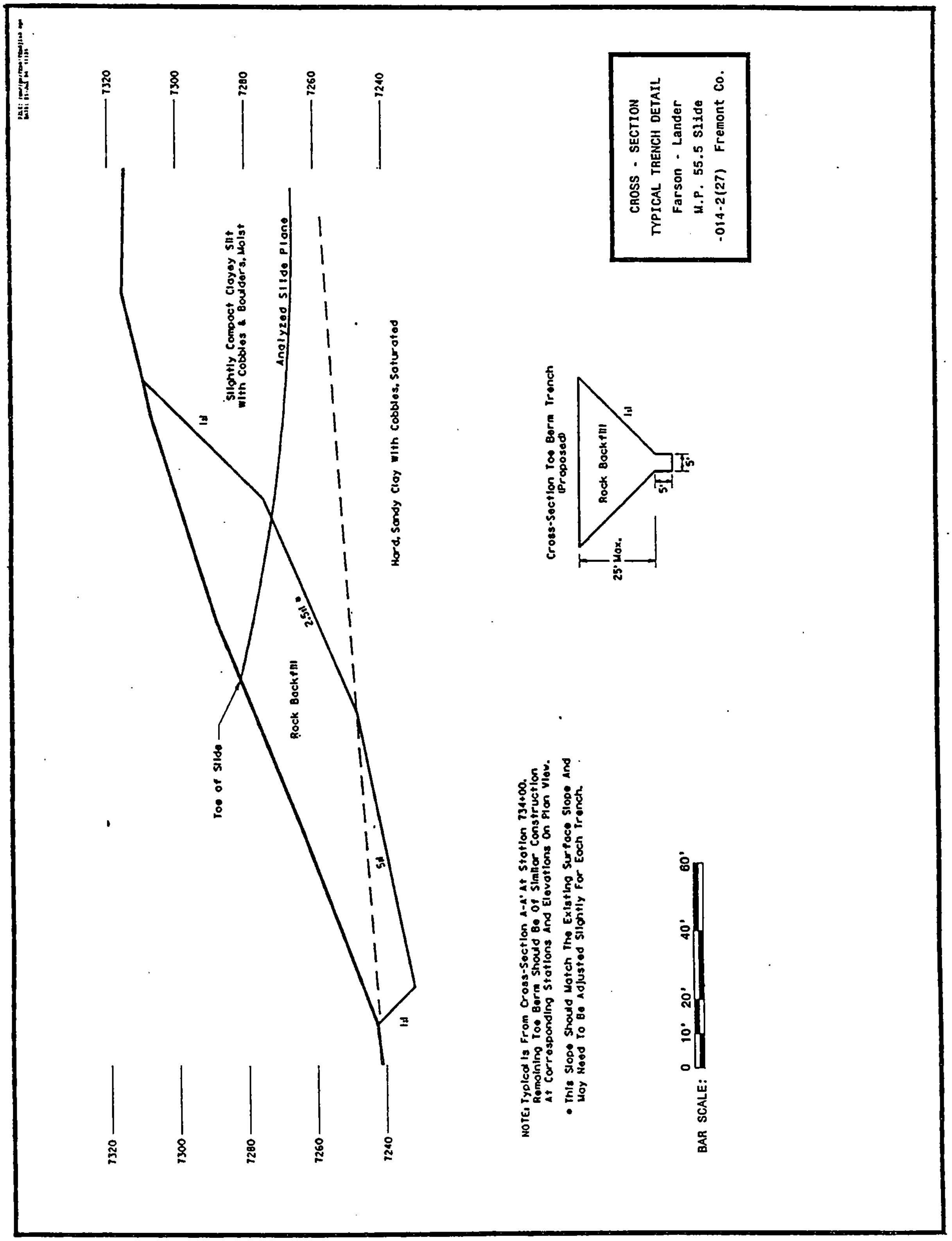

Figure 9. Cross section of typical toe trench detail.

SUMMARY

The contract was awarded on July 14, 1994 for $ 1,759,895.00 to Bartlett Inc., of Hanna, Wyoming, and construction is expected to start in August 1994. The contract will reconstruct 3,800 lineal feet of Highway 28 with a 76 foot maximum shift of centerline at M.P. 55.5. A 10 foot high fill will be constructed to match the existing grade utilizing 13,000 cubic yards of shredded tires to reduce driving forces and add stability to the existing embankment. The alignment shift will allow for the removal of some of the existing embankment to the desired fill slope configuration which created even greater increases in the factor of safety. The proposed fill slope will transition from a 3:1 to a 2:1 in the area of the spring, allowing for the construction of a three-sided, closed end concrete box over the spring. Four toe trenches will be constructed at the toe of the slide near the base of the hillside. The following table lists the bid tabulations for each of the recommended design changes.

MATERIALS AND RATES

Item	Quantity	Rate	Total
<u>CENTERLINE REALIGNMENT</u> 76′ maximum centerline shift	LUMP	LUMP	$555,880.00
<u>LIGHTWEIGHT FILL</u> Shredded tires	13,000 c.y.	$4.00/c.y. Tires $26.10/Tires & Haul	$339,300.00
<u>UNCLASSIFIED EXCAVATION</u> Removal of existing embankment	146,000 c.y.	$1.10/c.y.	$160,600.00
<u>SPECIAL EXCAVATION</u> Toe berm trenches	17,000 c.y.	(U.S. Steel Pit) $7.00/c.y. (iron ore)	$119,000.00
<u>CULVERT EXCAVATION</u> Three-sided, closed end box	7,160 c.y.	$8.50/c.y.	$ 60,860.00
<u>BOX CONSTRUCTION</u> Three-sided, closed end box	LUMP	LUMP	$105,950.00

Table. Tabulation of bid items from the low bid, Bartlett, Inc., Hanna Wyoming.

The construction sequence is critical. Slide repair will begin with the construction of the four toe trenches. Each trench will be built one at a time to maintain slide stability and will be approximately 155 feet long and spaced on 60 foot centers, in a V-type design. The rock backfill material will come from both the U.S. Steel Pit mine located up the mountain 4.92 miles and the Tensleep Sandstone rock cut, right of centerline at M.P. 55.7.

The next phase of construction will be the placement of the shredded tire fill. The use of tire shreds in place of a soil embankment will result in a weight reduction ranging from 50-75 percent or 10,000 to 15,000 tons. The lightweight fill will require the use of about 500,000 tires. A six-inch underdrain pipe will be installed below the tires, and separation fabric with two feet of select embankment will be placed over the top of the tires. The tire fill will be temporarily surfaced and act as a detour while the existing embankment is removed to the recommended fill slopes. The existing embankment material will be excavated to the planned fill slopes and the material wasted outside of the project limits. The plan is to have these first three steps completed before winter of 1994.

In the spring of 1995 the last two stages will be completed. The first of these phases will consist of exposing the spring and constructing the three-sided, closed end concrete box. The excavation slopes, which are needed to expose the spring at natural ground, will infringe on a small area of the detour. The box will be constructed into the side of the fill slope allowing the spring to flow into the bottom of the box and outlet at the end of the box located at the toe of the fill slope. The box culvert measures 10 feet high by 20 feet wide and 43 feet long. The final stage will be placement of the crushed base and paved surfacing section.

REFERENCES

Hausel, W. Dan, 1992, *Revised Geologic Map Of The Miners Delight Quadrangle, Fremont County, Wyoming*, Revised from: Bayley, R. W., 1965, U.S. Geological Survey GQ-460.

Hausel, W. Dan, 1991, *Economic Geology Of The South Pass Granite-Greenstone Belt, Southern Wind River Range, Western Wyoming.* The Geological Survey Of Wyoming, Report of Investigations No. 44. Gary B. Glass, State Geologist.

Siel, D. Barry, 1989, *Expanded Polystyrene As A Lightweight Fill For A Highway Embankment,* Abstract from the 15TH N.W. Geotechnical Workshop, October, 1989, Coeur d'Alene, Idaho.

CATASTROPHIC EMBANKMENT FAILURE SOUTH OF SHERIDAN, WYOMING INTERSTATE 90 - M.P. 41.1

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ABSTRACT

On June 9, 1993, minor cracks appeared in the southbound lane (SBL) of I-90, 17 miles south of Sheridan, Wyoming. The next day the site was inspected by Geology personnel of WYDOT and a geotechnical investigation initiated. By June 15, the cracks involved a 300 foot section of the driving lane and differential settlement was measured at 1 inch per day. The preliminary investigation was completed by 11:30 a.m. on June 22. At 12:30 p.m. movement accelerated to 2-3 feet per hour, and by 8:00 p.m. the roadway had dropped 30 feet and the SBL was closed to traffic and a detour set up. Total movement caused the lane to drop 50 feet and the slide toe was 600 feet from centerline. Over 150,000 cubic yards of fill was affected and a small drainage was dammed by the debris.

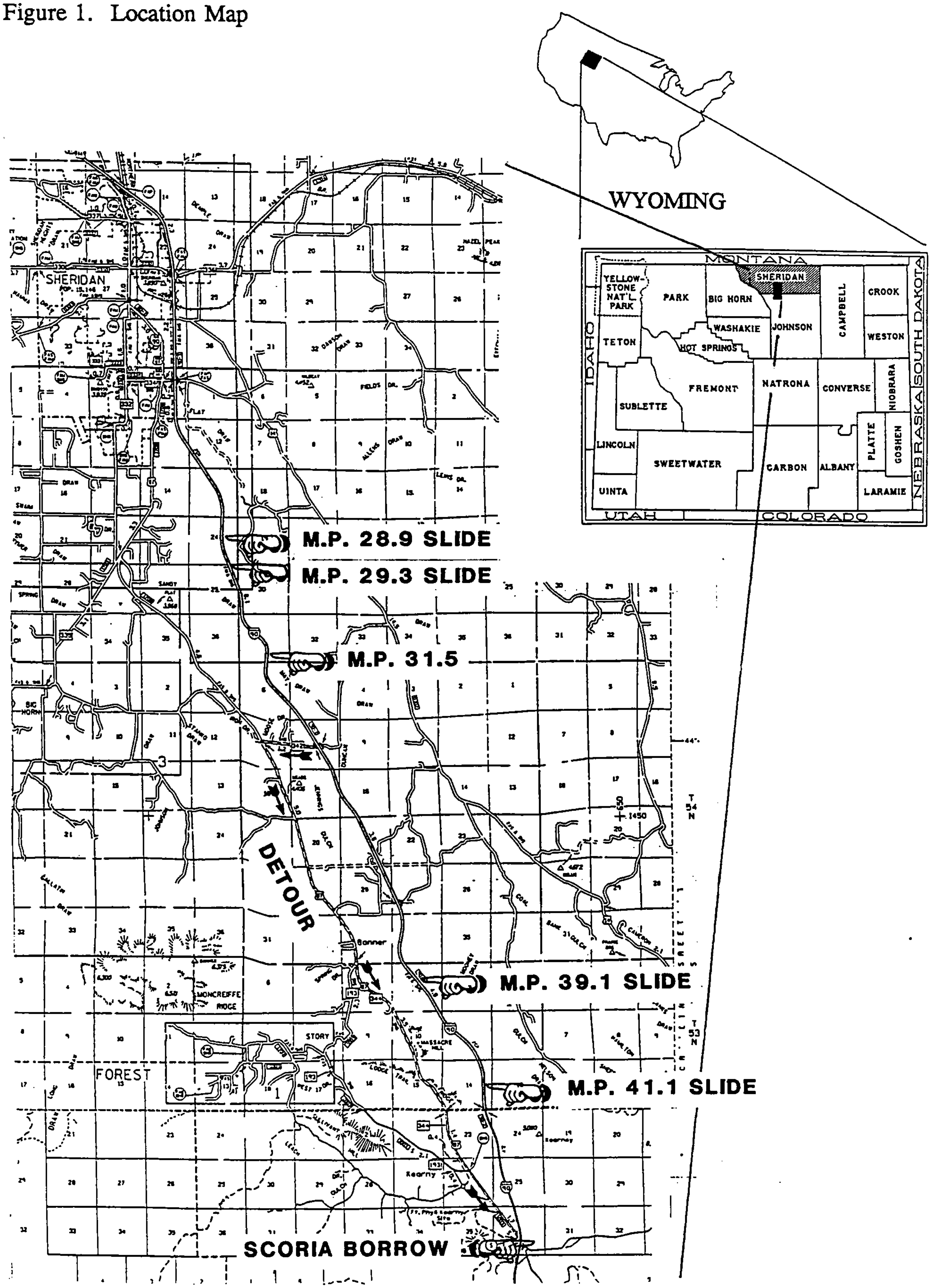
The slide occurred in a 100 foot high fill built in 1967 out of A-7-6 type soils. The geotechnical investigation in the toe area found the slide plane in all test holes and groundwater was not detected. Soil testing indicated the soil moisture was just above the plastic limit (19-21%). A slide analysis was performed using XSTABL, and various solutions were investigated. The most stable and economical recommendation was to remove the slide debris and rebuild the fill using better material (scoria). An emergency contract was let on August 6 for $691,000. The work was completed on November 11 and the SBL was open to traffic five months after the first crack appeared.

The cause of the failure was determined to be long term moisture buildup within the clayey fill, which resulted in the loss of shear strength of the fill. The heavy spring rains added enough additional weight that the fill could no longer support itself. Other states are having similar problems with high clay fills (50 feet+) built 20-25 years ago during the Interstate construction boom. WYDOT is currently researching and inventorying all high fills on our Interstate system to see how many more potential slides exist.

INTRODUCTION

Interstate 90 crosses northern Wyoming for a distance of 207 miles. It is one of the nation's main east-west arteries and connects Boston to Seattle. Figure 1 is a map showing the location of the slide relative to local geographic features. In Wyoming, I-90 crosses the Powder River Basin, which contains the highly dissected Tertiary Wasatch Formation. The Wasatch Formation is composed of interbedded shales, coals and sandstone. Some coal beds are over 200 feet thick. In the Buffalo-Sheridan area, the Wasatch consists of highly plastic clay shales and coal seams.

Figure 1. Location Map



The coal seams carry water and have also burned along exposed outcrops to form baked shale or "scoria" deposits. The existing topography is covered with recent landslides and landslide deposits.

During construction of I-90 in the late 1960's, many of these landslides were identified and were remediated by underdrain systems and removal of the slide debris. The dissected nature of the land forms and the design criteria developed for the Interstate systems combined to cause the construction of many 100+ foot high cuts and fills. The fills were constructed using standard compaction techniques utilizing the local soils and shale bedrock. In most cases, the fills were built below optimum moisture contents for constructibility.

On June 9, 1993, the Sheridan Maintenance crews noticed some small cracks in the driving lane of the SBL I-90 at M.P. 41.1

June 9, 1993
Failure Minus 14 Days

Gary Riedl's last afternoon as WYDOT Chief Engineering Geologist was spent at a retirement coffee held in his honor so that friends and co-workers could congratulate him and wish him well. At 3:00 p.m., Jay Gould, the District 4 Maintenance Engineer, phoned the Geology Program to report that Maintenance personnel had reported some cracks in the pavement and wanted a geologist to check them out. Gary was notified and then claimed "Not <u>my</u> problem". A State plane was scheduled to fly to northern Wyoming the next day, so Mike Hager arranged to fly to Sheridan.

June 10, 1993
F Minus 13 Days

A reconnaissance of the site was conducted by Geology and District 4 engineering personnel. A 200 foot discontinuous line of cracks was forming in the parking lane with minor cracks extending into the driving lane. The road surface within the cracked area was slightly deformed with up to 1 inch of deflection. Another continuous line of cracks was located at the base of the guardrail. The fill slope showed no signs of movement. A 500 foot row of lathe was lined up across the slope to help detect movement.

A review of the "As-Constructed" plans indicated that the fill was 45 feet high at lane centerline and over 100 feet high from the toe of the 2:1 fill slope. The road grade is about 1% and the median ditch section was very wet. Median drains are spaced every 1,000 feet with the nearest drain 300 feet up gradient from the cracks.

A drill rig was scheduled to arrive on site the next week.

June 14-17, 1994
F Minus 8-5 Days

WYDOT auger drill rig H-823 arrived on site to drill test holes through the road surface. Five test holes were planned, three in the road and two in the median. The maximum depth reached was 73 feet and 2 inch perforated plastic pipe was placed in each hole to measure the groundwater surface.

The Maintenance Foreman had set targets and was measuring the drop in the road surface each day. His measurements were averaging a drop of 1 inch per day. There was still no visible distortion of the fill slope, and Maintenance had set up barrels to close off the driving lane.

June 22, 1994
Failure Day

The WYDOT drill rig was on site in the morning. (See Figure 2.) The cracks were affecting 300 feet of the road surface and the parking lane had dropped about one foot. The guardrail was visibly deflected and the cracks could be traced 5-10 feet downslope on the fill face. The right-of-way (ROW) fence at the base of the fill was starting to tilt over.

At 11:30 a.m., the preliminary drilling operations were completed and the drill rig was sent to Sundance, Wyoming, where another slide was affecting the road on Oudin Hill near Devil's Tower. Mark Falk and the WYDOT ATV drill rig 4001 were at M.P. 28.9 on I-90 drilling another slide that has been slowly failing since 1986. Movement on this slide had accelerated this spring and it was expected to fail soon. Crossovers to divert traffic had been built earlier and Maintenance was prepared to close the road and shift to two-way traffic on the northbound lane (NBL).

Back at M.P. 41.1, at 12:30 p.m. the slide movement accelerated from dropping 1 inch per day to 2-3 feet per hour. Maintenance mobilized a crew to set up portable jersey barrier for safety and to keep the SBL open to one lane. A pressure ridge had formed at the toe at the base of the fill. By 6:00 p.m. the slide debris had dropped 20-30 feet and the road surface beneath the jersey barrier was starting to drop. Maintenance crews decided to close the SBL and divert traffic to the old highway which parallels I-90. This detour forced the southbound traffic onto a narrow two-lane highway for 12 miles. At 8:00 p.m. the slide scarp had migrated to centerline of the travelway and the jersey barrier had fallen into the slide. The box beam guardrail was still intact and hanging in midair above the slide with all its support posts attached. The toe of the slide was now past the ROW fence and threatening to dam off a small drainage. (See Figure 3.)

June 23, 1994
Failure Plus 1 Day

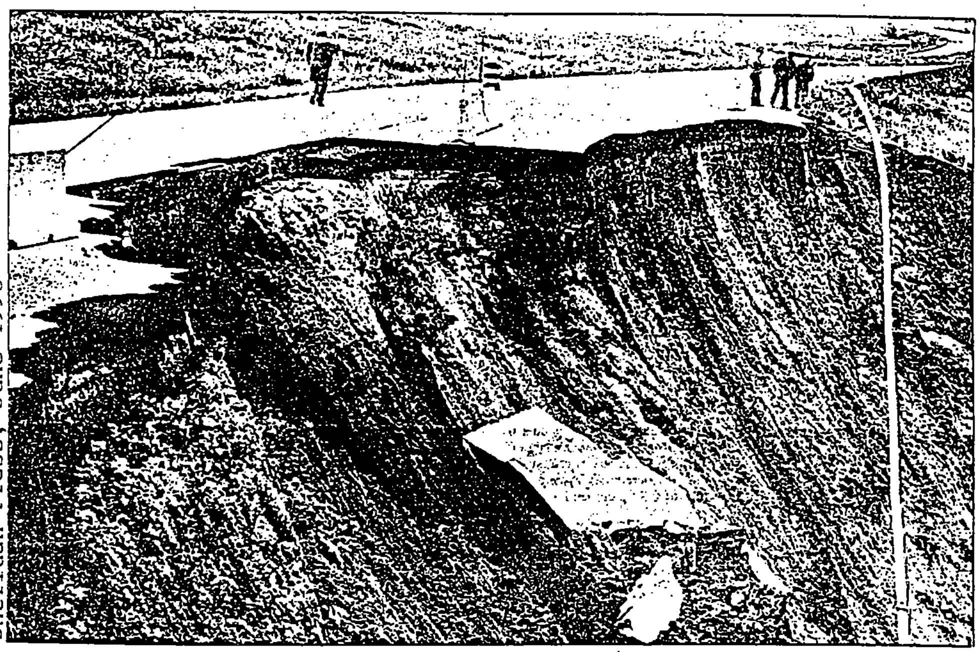
The District 4 traffic engineers determined that the emergency detour would not be adequate as a long term detour and started designing crossovers on I-90 near the slide to move traffic onto the NBL. Two crossover points were chosen about two miles apart and a local contractor was contacted to build and surface the proposed crossovers.

The local newspapers in Wyoming and Montana covered the story of the slide failure and the closing of I-90. This helped notify the traveling public of the problem and necessity for detours. (See Figure 4.)

Figure 4. Newspaper Article

—Interstate exits

Sheridan Press, June 1993



Mud slide damages interstate

Traffic being detoured; repair plans under way

Compiled from Press and AP reports

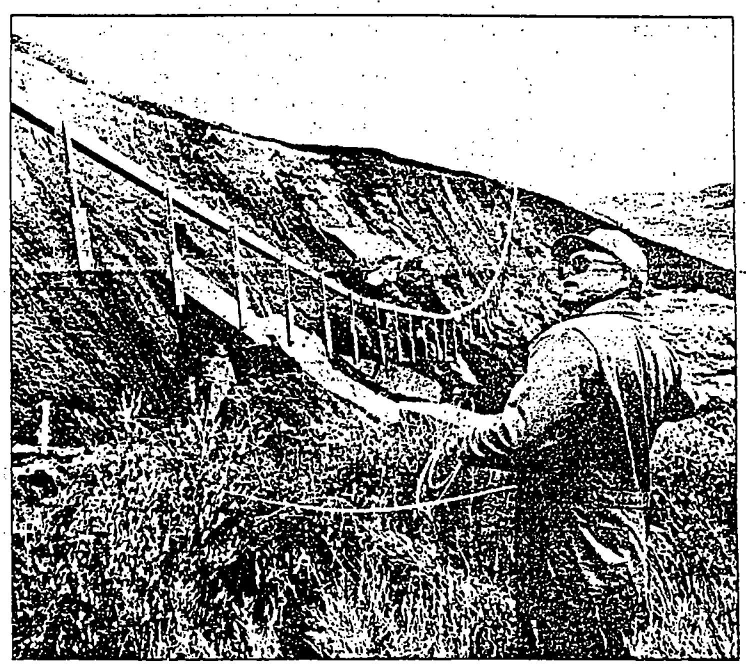
Heavy spring rains have caused a large chunk of Interstate 90 south of Sheridan to slide downhill Tuesday, officials said.

Traffic for the southbound lane of I-90 north of the border between Johnson and Sheridan counties is being rerouted to nearby U.S. Highway 87 at exits milepost 33 and 42, according to the Wyoming Highway Patrol.

Jay Gould, district maintenance engineer for the Wyoming Transportation Department in Sheridan, said the ground slipped because it was saturated from heavy spring rains. No injuries were reported.

The department had been watching the area in recent days and noticed it starting to slide two feet per hour Tuesday afternoon, he said. A big section then broke down the 25-foot cliff about 3:30 p.m. Tuesday.

Gould said approximately one lane of the interstate for 300 to 400 feet broke loose. He said the other lane shows "instability" so officials decided to reroute traffic.



ROAD HAZARD — TOP: A 300 to 400 foot section of the southbound lane of Interstate 90 about 17 miles south of Sheridan gave way Tuesday prompting highway department officials to close off the highway. Traffic is being rerouted onto U.S. Highway 87 from I-90 exits 33 to 42. ABOVE: Jack Bare of the Wyoming Department of Transportation surveys the damage from the bottom of the slide area Wednesday morning. The suspended guardrail is visible where it extends across the slide area. (Press photos by Mark Slupe)

The northbound lane some distance away was not affected by the slide, he said.

Gould said a detour is set up at the Meade Creek exit, and southbound traffic is allowed back on the interstate at the Piney Creek exit near Lake DeSmet.

He said the transportation department plans to run two-way traffic on the northbound lane until the road is fixed, but must first construct "crossovers" and signage. The crossovers will be in place by late this week or early next week, he said.

Transportation department crews will complete some of the work, but contractors will have to be hired for part of the construction, Gould said.

June 28, 1994
F Plus 6 Days

WYDOT drill rig 4001 had completed its work at M.P. 28.9 and was moved to M.P. 41.1. A district bulldozer was brought in to build access trails for the drill rig. Eight more drill holes were completed at various points on the slide in order to locate the slide plane and obtain samples for testing. The slide was still moving during the drilling operations and by the time drilling was complete, the original road surface had dropped 50 feet. The test holes had sheared off at the slide plane, so measuring the depth to the slide plane was very accurate. The toe of the slide had crossed the drainage and a small lake was formed.

June 30, 1993
F Plus 8 Days

The contractor completed the crossovers and they were paved and striped. All detour signing and two-way barrels were set up to move all the traffic to the NBL, and the emergency detour signing was removed. The Photos and Surveys Program of WYDOT was setting up ground control so that the slide area could be photographed from the air. The wet spring had produced knee-high grass in the ROW, which required Maintenance to mow the ROW so that targets could be placed on the ground and to get an accurate view of the ground surface. Preliminary air photos were taken on June 30 and the survey photos were taken on July 2, 1993. (See Figure 5.)

July 5-18, 1993
F Plus 13-26 Days

The Geology Program began its office phase of the geotechnical investigation. Preliminary field survey information was plotted to produce plan views and cross-sections of the slide. The geotechnical soils foundation lab began testing the soil samples. Classification, direct shear and residual direct shear tests were performed on the samples. The fill was constructed out of A-7-6 (CL-CH) type soils which had moisture contents from 13% to 45% and had plastic limits between 15-30.

A slide analysis was performed using XSTABL, and various types of solutions were investigated. Figures 6 and 7 show the plan view and cross-sections of the slide. Some of the solutions involved lightweight fill material, rebuilding the fill utilizing shredded plastic fibers, and realigning the road. Two final solutions were proposed and evaluated for stability, constructibility and costs. The first involved removing most of the slide debris and rebuilding the 2:1 fill with granular material. The second alternative recommended removing less slide debris and reconstructing the fill to a 1.5:1 slope utilizing a geogrid to reinforce the steepened slope. An economic cost analysis favored the first alternative. With expense of hauling scoria to rebuild the fill, a slide analysis was performed with a fill topped off with 20-30 feet of clay.

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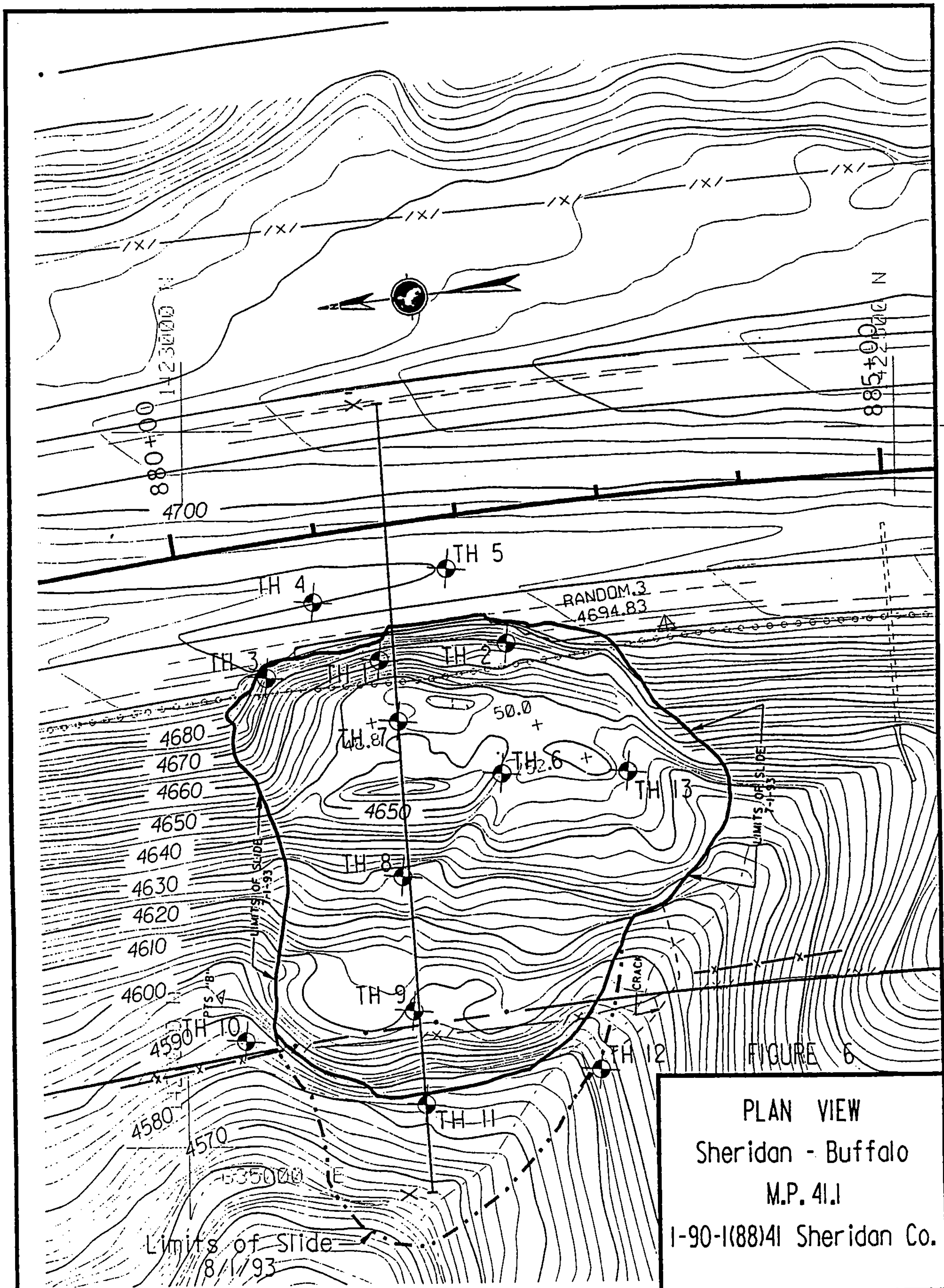

FIGURE 6

PLAN VIEW
Sheridan - Buffalo
M.P. 41.1
I-90-1(88)41 Sheridan Co.

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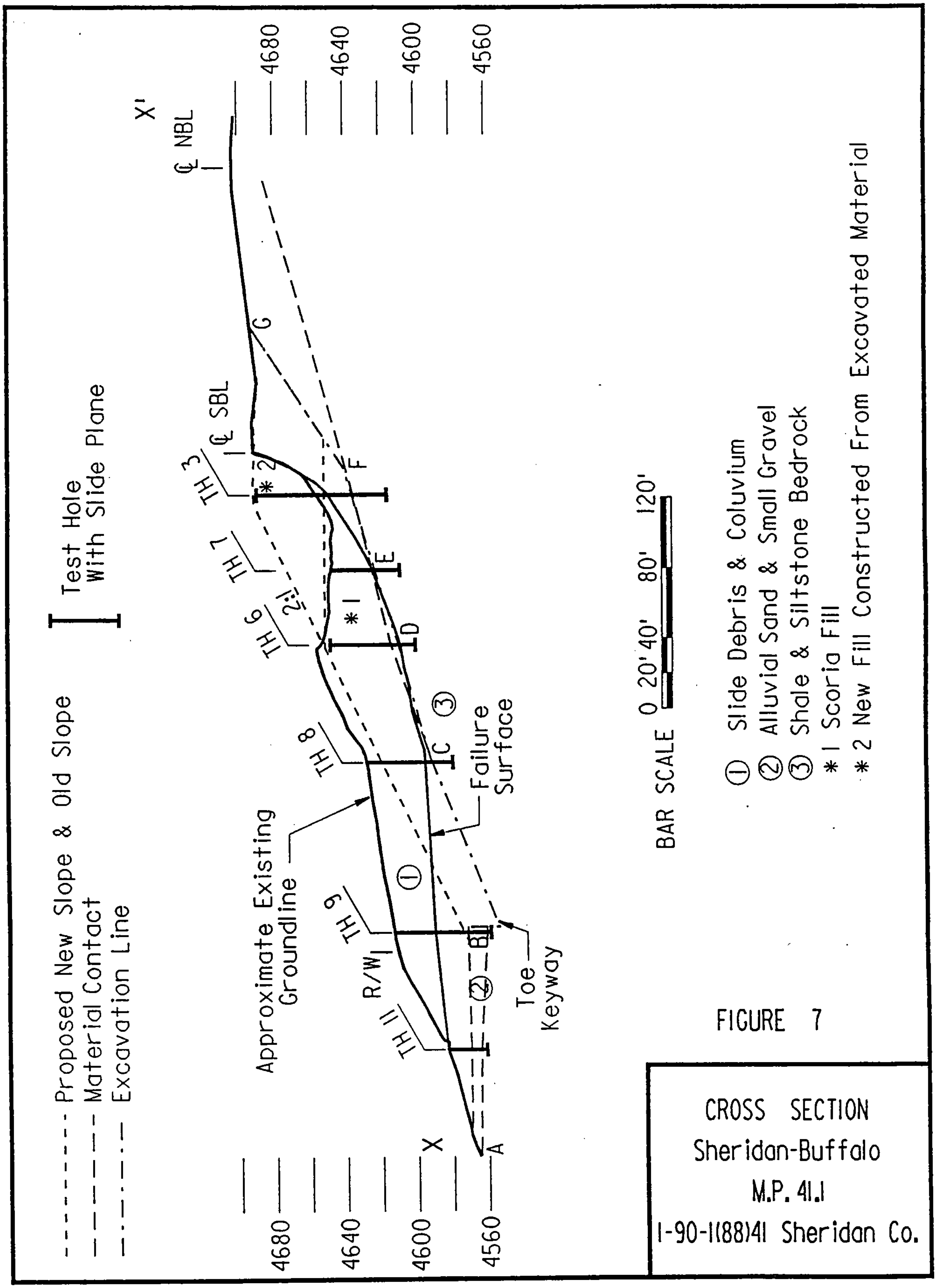

FIGURE 7

The parameters used in evaluating the slide are listed in Table I. Also included are parameters from three other fill slides on I-90. The slide at M.P. 29.3 was caused by placing a 10 foot thick sliver fill on an existing 100 foot high embankment to widen it to meet design standards. An analysis of this slide revealed that the addition of the sliver fill decreased the factor of safety by 1.5%, causing another major fill failure.

TABLE I
SLIDE PARAMETERS

Slide M.P.	Soil Type	Unit Weight	Cohesion	Degree Phi	% M	Liquid Limit	P.I.
41.1	A-7-6	125	500	5	25	61	33
39.1	A-7-6	124	570	9	?	51	26
29.3	A-7-6	124	838	9	25	57	33
28.9	A-7-6	124	838	9	24	52	32

July 19-28, 1993
F Plus 27-36 Days

The final geology report was submitted to the District engineering personnel for review. A source of scoria was located within three miles of the slide and arrangements were made to purchase the material from the landowner. Fortunately the scoria borrow area and the land outside the ROW affected by the landslide were both owned by Texaco, Inc. Final quantities were estimated and a set of bid plans produced. The project was advertised on July 28, 1993.

August 6, 1993
F Plus 45 Days

A total of three bids were submitted and opened on August 6. A telephone conference call was set up between all the members of the Transportation Commission, and the bid was awarded to E.H. Oftedahl and Sons of Miles City, Montana for $690,813.25. The cost for the unclassified excavation bid item was $2.00 per cubic yard. The original plan quantity was 181,000 cubic yards.

August 12, 1993
F Plus 51 Days

The Notice to Proceed was given to the contractor and they started mobilizing the next day. Equipment on site included four scrapers, two bulldozers, a front end loader and a grader. Twelve belly dumps were added later to haul the scoria from the borrow area. A small fill slide 1600 feet south of the main slide was also included as part of the construction contract.

September 15 - October 27, 1993
F Plus 85-127 Days

The toe area was filled with compacted scoria and the rest of the fill was placed on a 2:1 slope. Due to the expense of hauling the scoria from the borrow area, it was decided to complete the top 30 feet of the fill utilizing the soil from the waste area. This would speed up the placement of fill and get the fill completed in time to have the road surfaced before winter. A total of 87,000 cubic yards of scoria and 55,000 cubic yards of clay fill was used to rebuild the fill. Placing of the fill averaged 4,000 cubic yards per day. The landowner (Texaco) requested that the pond formed behind the slide toe remain and the slide debris formed into a small dam. A small overflow ditch was cut onto the side of the dam.

October 28 - November 11, 1993
F Plus 128 - 142 Days

With the fill completed, the subgrade was blue topped and prepared to place the crushed base. After the crushed base was placed, the asphalt surfacing was laid. New guardrail was installed along the shoulder and the road surface was striped.

On November 11 the detours were removed and traffic returned to the SBL of I-90, four and one half months after the slide occurred. The slide caused an emergency compression of the geotechnical investigation, design, plan preparation, bidding and final construction. This could not have occurred without the complete cooperation of all State, Federal and private parties involved.

THE FINAL ANALYSIS

The cause of the slide can be attributed to a few factors, which when combined, lead to the ultimate failure of this high fill. These factors include:

1. <u>Poor soils.</u> The poor soils in which the fill was constructed appear to have adequate shear strength when placed in a below optimum moisture condition, but over time the clays absorb water and lose strength. However, the shale hills were the only materials available that could be economically used to build the fills in this area of Wyoming.

2. <u>Design Criteria</u>. The design parameters for Interstate highways at the time of construction were conservative and based on higher design speeds. This forced the grades and alignment to be very flat and straight. This causes heavy grading and produces high fills and deep cuts.

3. <u>Time.</u> This section of Interstate was built 25 years ago and it appears that it takes that long for the soils to absorb enough water to reach the point where the soil can no longer support its own weight.

The event which finally triggered the catastrophic failure was the abnormally wet spring which saturated the top portion of the fill and added enough weight to overcome the shear strength of the clay.

PROSPECTS FOR THE FUTURE

There have now been four major fill failures within a 20 mile stretch of I-90 in Wyoming. Another fill at M.P. 31.5 has cracks in the road and is being monitored with an inclinometer. Within this 20 mile section of Interstate are 20-25 fills that are over 50 feet high. I-90 is built on the Wasatch Formation for another 90 miles on similar topography. Interstates 25 and 80 in Wyoming cross geologic formations and topography that have the right conditions to produce more fill failures on 80 more miles of Interstate. This potential for catastrophic fill failures in Wyoming is disturbing and has been reported to the WYDOT Administrative and District Engineering Staffs for consideration in budget planning. At this point, the Transportation budget can handle one major fill failure per year; any more than that will impact the construction budget. The Geology Program is now working with Project Development in designing projects that require fill widening. The project currently under construction at M.P. 28.9 has proven just how close to failure these fills are. The addition of a 10 foot sliver fill lowered the safety factor 1.5% and lead to another fill failure.

During the geotechnical analysis of this slide, other states were contacted to see if they were having similar problems. Texas, Louisiana and Arkansas reported similar problems but on a smaller scale. In May of 1994 an E-mail questionnaire was sent to all AASHTO member states requesting information on Interstate fill failures on embankments over 50 feet high. To date, thirteen states have responded, with Illinois, Kansas, North Dakota and Kentucky reporting fill failures.

ACKNOWLEDGEMENTS

The authors would like to thank Connie Fournier and Marilyn Foster for typing and editorial assistance and Jim Coffin and Jim Dahill for reviewing the manuscript. They would also like to thank all the engineers and maintenance personnel in District IV for all their help during the investigation and construction phases. Finally to Cindy Hager and Debra Falk for their love, patience and understanding while we are out in the field.

REFERENCES

Love, J.D. and Christiansen, A.C., 1985, *Geologic Map of Wyoming*, U.S. Geological Survey and Wyoming Geological Survey.

Interactive Software Designs, Inc., Copyright (C) 1992, *XSTABL, v. 4.103.*

Development of a Time Domain Reflectometry System to Monitor Landslide Activity

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ABSTRACT

Time domain reflectometry (TDR) was originally developed to find breaks in power transmission cables. This testing method uses the characteristics of a returned pulse to monitor the shear or rupture of a coaxial cable. The use of TDR to measure rock mass deformation is relatively new. Some applications were successful in the mining industry to monitor strata movement. The capability for remote access of the data exists. Economic advantages of such a system over the traditional inclinometer method include: no need to physically visit the site to collect data; readily available, inexpensive, and disposable cable; no need to clean debris from the inclinometer hole before taking measurements; and the ability to monitor up to 512 holes with one tester and multiplexer hook-up.

As a field test of a prototype system, the California Department of Transportation (Caltrans), the University of the Pacific, and Neil O. Anderson and Associates installed a coaxial cable in a large landslide along U.S. Highway 101 near Crescent City, California. The area is being studied to develop roadway stabilization alternatives. The landslide is occurring in rocks of the Franciscan Complex. In the area of the roadway it moves on the order of 1.5 to 3.0 m/yr (5 to 10 ft/yr). Two inclinometers were installed in the slide to determine the depth to the failure plane(s). One of the inclinometers is located in the roadway, and requires traffic control before it can be read. The coaxial cable of the TDR system was strapped to the outside of this inclinometer's casing to allow for a comparison of the data.

The initial data show a spike in the TDR reading at approximately the depth where the inclinometer casing is deflected. This suggests that TDR may be used in place of an inclinometer in some instances.

INTRODUCTION

Time domain reflectometry (TDR) is a method of analyzing electrical signals for testing purposes. It originally was developed to find breaks in power transmission cables (Franklin and Dusseault, 1989). This testing method uses characteristics of a returned pulse to determine the amount of strain, or a rupture, in a coaxial cable.

Although developed for cable testing, TDR is finding use in geotechnical applications, especially mining (O'Connor and Wade, 1994). For example, a study by the U.S. Bureau of Mines determined the height of rock caving above longwall coal mines (Dowding, et al., 1989). Coaxial cables embedded in bore holes prior to mining were used to infer deformation and collapse in the overburden. This information gave an indication of the extent of caving and shearing, and the associated bending of rock strata.

Currently, the California Department of Transportation (Caltrans) uses inclinometers to measure landslide movement and occasionally uses wire extensometers as failure warning systems. The inclinometer is a probe manually lowered down a specially cased borehole drilled into the slope. Accelerometers are used to monitor the orientation of the probe as it moves down the hole. Changes in orientation over time indicate slope movement; rapid changes can indicate imminent failure. The primary disadvantage with the inclinometer is the necessity of a site visit by a technician to take readings. In contrast, the wire extensometer is placed across the head scarp, or some other prominent feature, assuming that the movement measured can be used to predict a catastrophic failure. The problem with this system is that it can be triggered by birds, deer, or falling tree limbs (Cann and Steiner, 1992).

Caltrans investigated the Last Chance Grade landslide along U.S. Route 101 in Del Norte County, California, Figure 1. The investigation included drilling, bore hole logging, geologic mapping, and developing recommendations for stabilizing the highway. As a part of the field work for this investigation, inclinometer casing was installed in two borings. A coaxial cable was secured to the outside of one of the casings for comparison of TDR and inclinometer data.

There are three recommended alternatives for stabilizing the roadway. Knowledge of the slide plane(s) location will be essential in selecting one of the roadway stabilization measures. The first alternative is to realign the highway in a tunnel excavated behind the slide plane. Alternate two is to realign the roadway slightly, stabilize the material below the roadway with a soldier pile and tieback wall, and stabilize the material above the roadway with several rows of slope stressing. The third choice is to realign the highway in a cut excavated behind the slide plane. The first option is expected to have the least environmental impact and the third, the most. Combinations of these alternatives could also be used to stabilize the roadway.

SITE DESCRIPTION AND GEOLOGY

This section of U.S. Route 101 was constructed on the west-facing flank of a 300 m (1,000 ft) high ridge. It is bounded on the west by the Pacific Ocean and on the east by Wilson Creek. The elevation of the roadway at this site is approximately 215 to 260 m (700 to 850 ft). Most of the area is covered with a dense growth of redwoods, douglas firs, and alders with a thick undergrowth of ferns and berry vines. The site is underlain by interbedded shale, sandstone, and conglomerate of the Franciscan Complex. These rocks are intensely fractured, sheared, and weathered to a depth of 15 m (50 ft). A major joint set strikes parallel to the ridge and dips 40° to 50° toward the ocean.

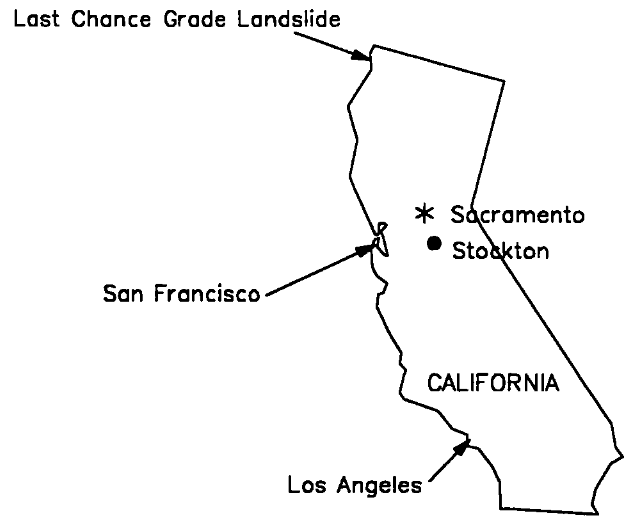

Figure 1 Location of Last Chance Grade Landslide.

Superimposed on the west-facing flank of this ridge is a large landslide complex. The slide complex is at least 915 m (3,000 ft) wide and 550 m (1800 ft) long in plan view. The slide complex appears to be a number of translational/rotational slides and debris flows that have coalesced. The highway crosses the upper portion of the complex.

The Last Chance Grade landslide, in the northern portion of the slide complex, is approximately 520 m (1700 ft) wide and 460 m (1500 ft) long in plan view, Figure 2. The slide is very active, affecting approximately 235 m (775 ft) of the roadway. This active area is composed of at least three translational/rotational slides (map units Qals1, Qals2 and Qals3 in Figure 2) with a debris flow (Qadf) snaking up the middle. The two lower units (Qals1 and Qals2) move as the ocean and rain erode the toes of these slides. As the two lower slides move forward, the upper slide (Qals3) is left unsupported and moves down behind the other slides. The three slides move as intact masses. During the rainy season soil, rock fragments, downed trees, and other material in the debris track flow downhill toward the ocean.

The southern portion of the slide complex is dormant. This conclusion was based on the fact that no evidence of recent movement (fresh scarps, bulges, or downed trees) was found during the field mapping. Eroded and tree-covered head scarps, side scarps, and closed depressions were used to map the dormant translational/rotational landslide (Qdls). Other eroded and tree-covered topographic features were interpreted as dormant debris flow tracks (Qddf).

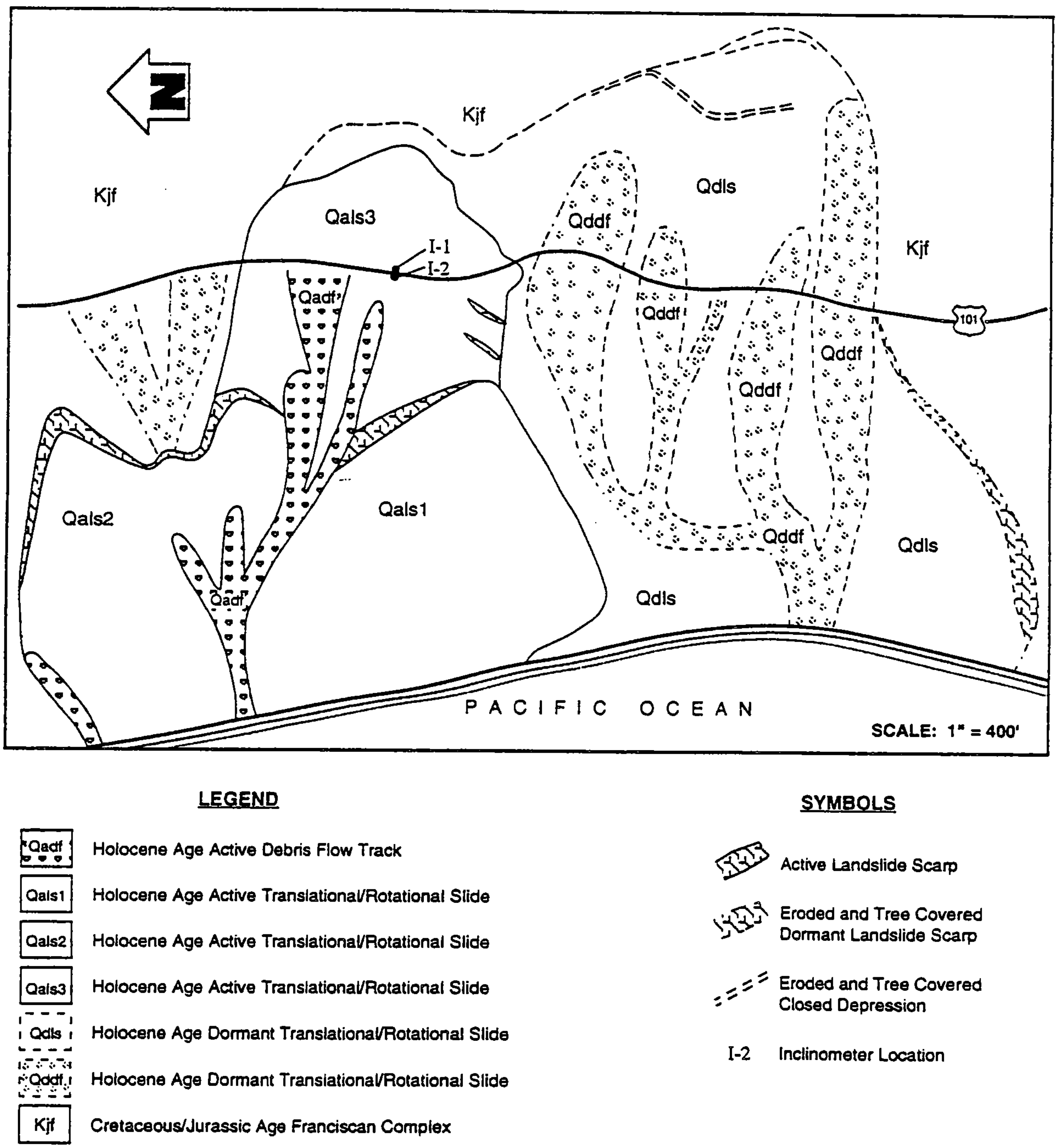

Figure 2 Geology of Last Chance Grade Landslide. I-1 and I-2 indicate slope inclinometer locations.

Two core borings were drilled into the center of the active slide, along the roadway, to obtain samples of the slide mass. Inclinometer casing was installed in the borings (I-1 and I-2 on Figure 2) to allow the determination of the depth to the slide plane. The coaxial cable for TDR measurements was installed in borehole I-2.

TIME DOMAIN REFLECTOMETRY

TDR is an electrical pulse testing technique where a cable tester, connected to a coaxial cable installed in a borehole, emits a stepped voltage pulse, Figure 3. Rock mass movements deform the cable, changing the cable capacitance and the reflected waveform of the voltage pulse. The time delay between a transmitted pulse and the reflection from a cable deformity determines the damage location. The sign, length, and amplitude of the reflected pulse defines the type and severity of the cable deformation (Dowding, et al., 1989).

The cable tester can be connected to a datalogger which records and stores reflections. The datalogger controls the cable tester and supplies power during measurements. The data then can be collected by computer automatically from a remote location using telecommunications

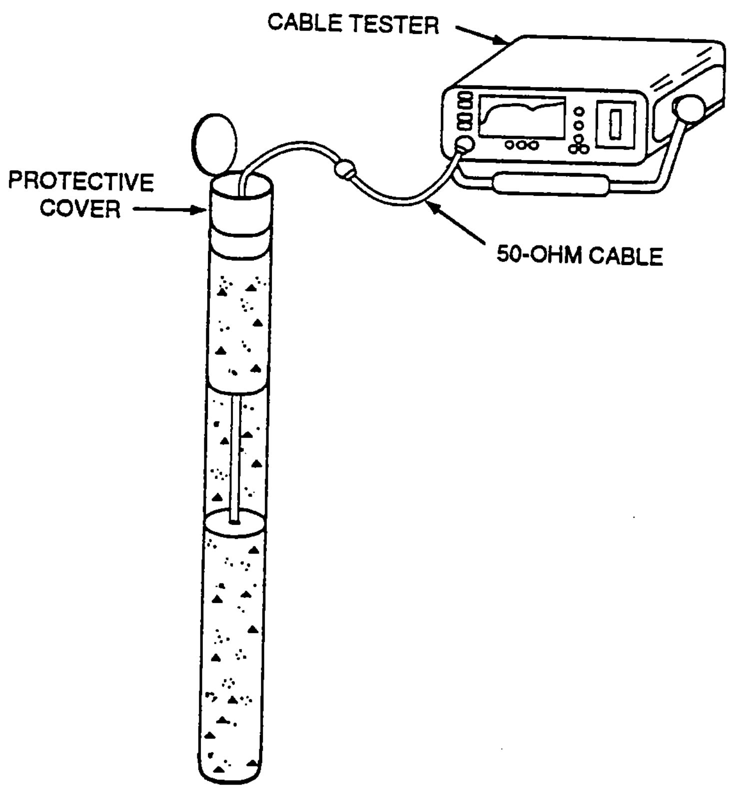

Figure 3 TDR installed in borehole (Dowding and Huang, 1994).

such as a phone or radio as shown in Figure 4 (Dowding and Huang, 1994).

The data consists of a series of TDR signatures. Different wave reflections are received for different cable deformations. The length and amplitude of the reflection indicates the severity of the damage. A cable in shear reflects a voltage spike which increases in direct proportion to shear deformation. A distinct spike occurs just before failure. After failure, a permanent reflection is recorded. In tension, the wave reflection is a subtle, trough-like voltage signal that increases in length as the cable is deformed. At failure a small necking trough is visible, which is distinguishable from a shear failure (Dowding, et al., 1989).

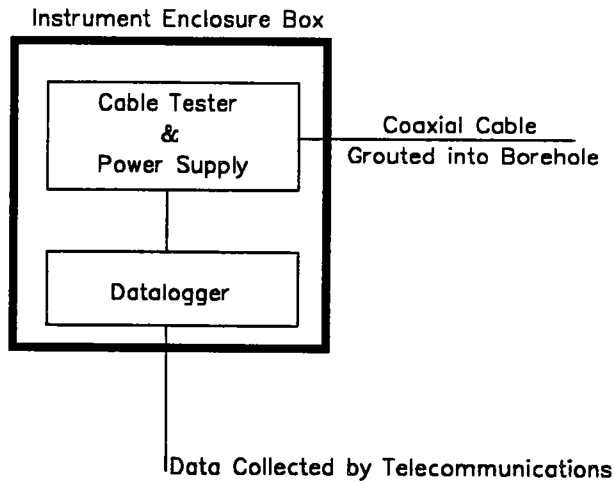

Figure 4 Schematic of a TDR installation.

By using the wave reflection data, rock mass movement can be monitored. As the amplitude of the wave reflection increases, zones of possible failure can be predicted.

TDR INSTALLATION AND RESULTS

A coaxial cable, RG 59, commonly used for videocassette recording, was attached to the outside, upslope side, of an inclinometer casing with nylon ties spaced approximately every 1.5 m (5 ft). The cable was installed in inclinometer hole I-2, 82 m (270 ft) of cable were used. The hole was backfilled with coarse aquarium sand, and a tremie pipe was used to flood the hole and compact the sand. A groove was cut in the asphaltic concrete overlay and extended beneath a K-rail barrier. The cable was laid in the groove and a BNC screw cable connector was attached to the end. The cable and connector were run beneath the K-rail, safely away from traffic, and placed in a plastic bag for moisture protection. The groove and the top of the inclinometer hole were backfilled with cold patch. An initial reading was taken on the cable, Figure 5. Four days later an initial inclinometer reading was performed, Figure 5. Twenty-eight days after inclinometer installation, an offset of about 23 mm (0.9 in) occurred at a depth of approximately 38 m (125 ft) in the N78W direction. Movement of the slide continued. Sixty-six to seventy days after installation, the casing moved upward 102 mm (4 in) relative to the road surface, leaving the inclinometer casing protruding. The top of the casing was cut off to allow for the smooth flow of traffic and the inclinometer data was reinitialized, beginning 74 days after installation. The inclinometer readings, therefore, indicated that the slide plane was approximately 40 m (132 ft) below the roadway at the center of the slide, Figure 6.

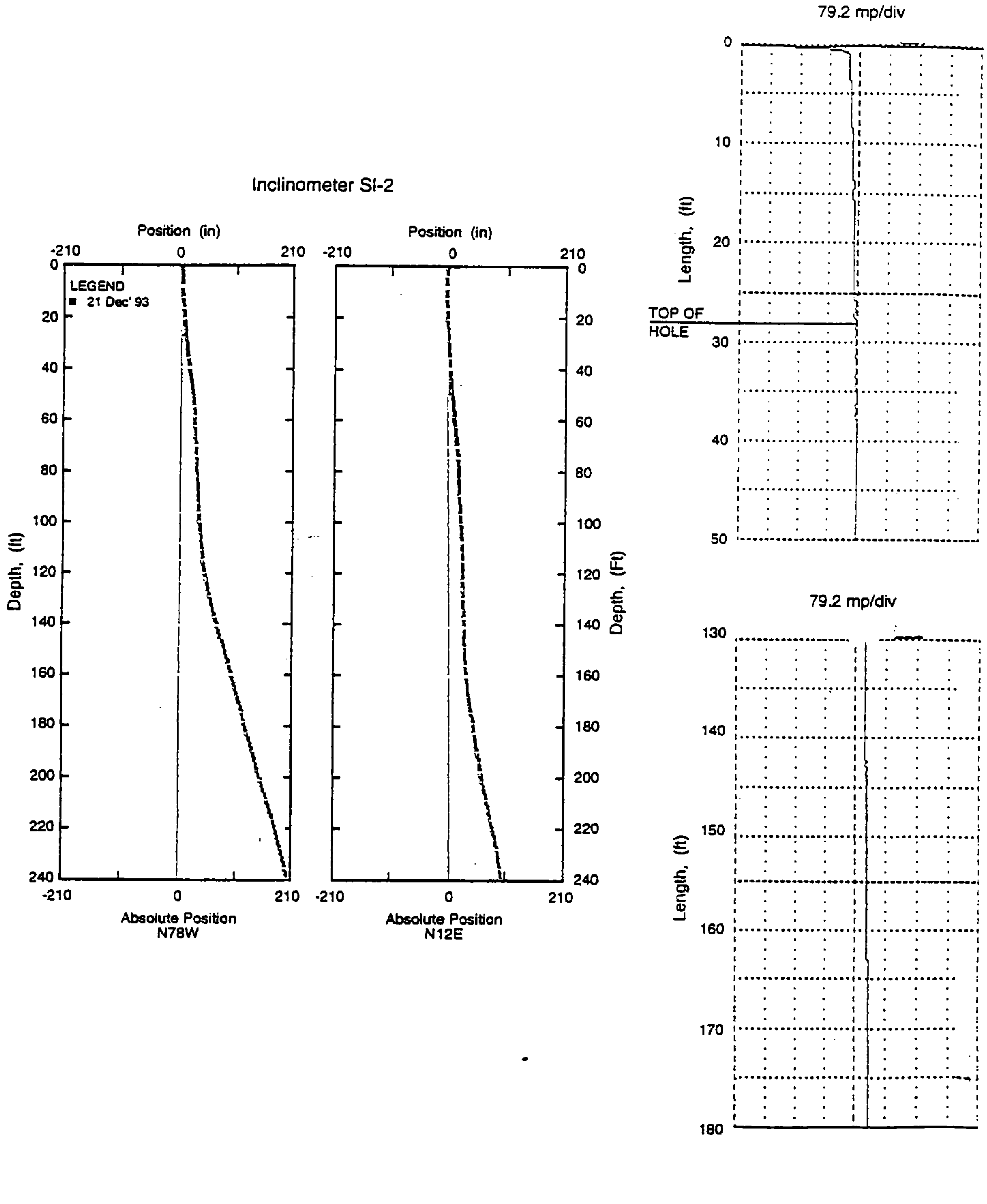

Figure 5 Comparison of initial inclinometer (I-2) and TDR readings (0 to 50 ft and 130 to 180 ft).

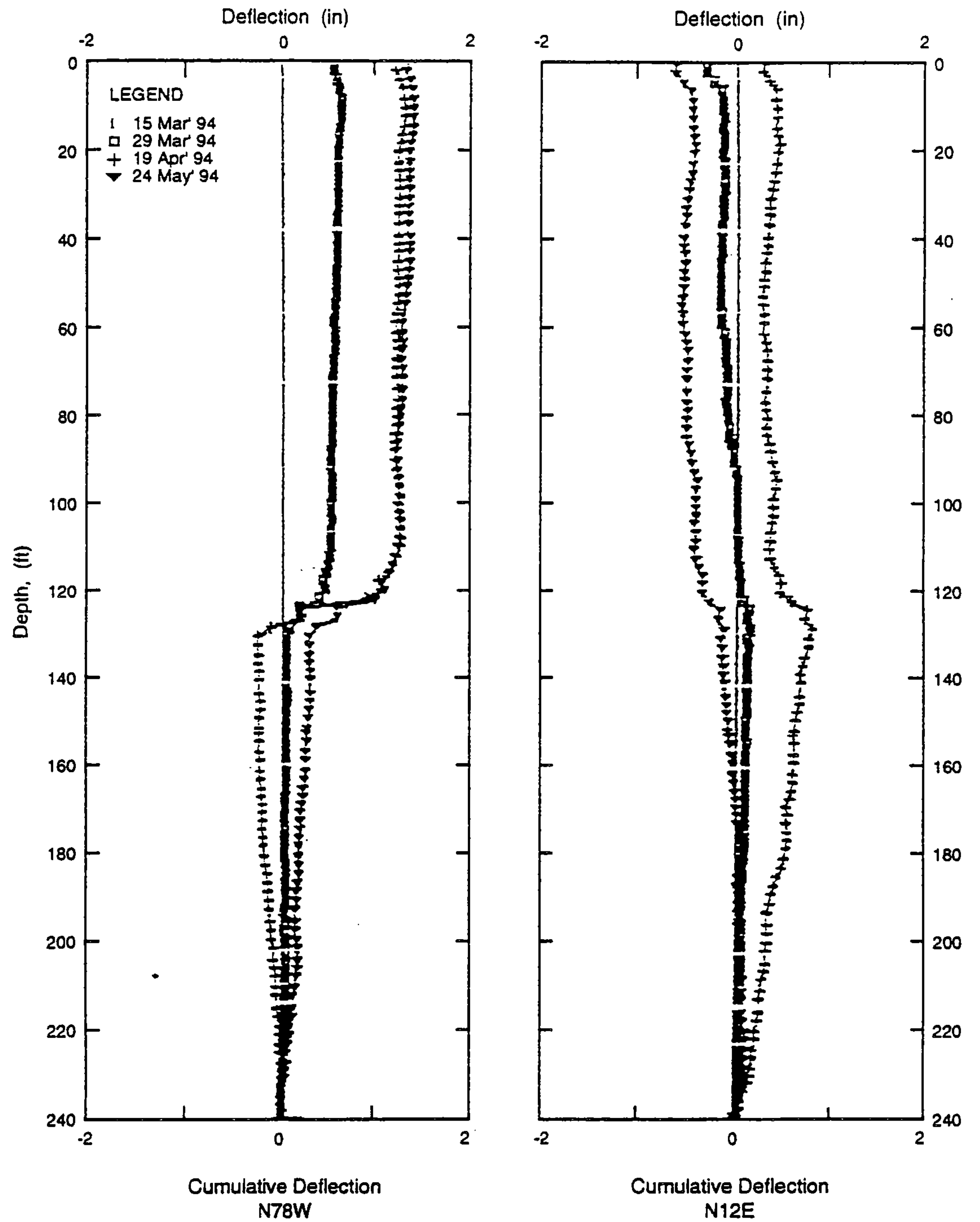

Figure 6 Output from inclinometer I-2 indicating slide plane at about 40 m (132 ft).

The TDR cable was read 148 days after its installation. The reading, Figure 7, showed two notable spikes. One spike, at 8.6 m (28 ft) indicated a crimp in the cable, probably where the cold patch squeezed the cable against the lip of the casing where it was bent to go down the hole. At a depth of approximately 40 m (132 ft), roughly the same depth as the inclinometer deflections, a trough in the signature was visible. The interpretation of this reading was that it indicated distress of the cable at the slide plane.

In order to read the inclinometer, it was necessary to implement traffic control for the safety of personnel. The TDR cable was read from behind the K-rail. No traffic control was necessary and personnel were protected from vehicular traffic.

CONCLUSIONS

Initial results of this study indicated that TDR may be used instead of inclinometers for landslide monitoring in some situations. A correlation between TDR signature and slide plane, as indicated by the slope inclinometer, appeared to exist. Reading data from the TDR cable was accomplished by safely standing behind the K-rail barrier.

The technology exists to remotely access the TDR installation. Caltrans, the University of the Pacific, and Neil O. Anderson and Associates are in the process of installing such a system at the Last Chance Grade. The advantages of such a system will be:

- No need to physically visit the site and, therefore, no traffic control is necessary
- No need to clean debris from the inclinometer casing before taking measurements
- Readily available, inexpensive cable
- Ability to monitor up to 512 holes with one tester and multiplexer hook-up

If successful, this will result in a cost savings to Caltrans in person-years (PY), possibly up to 6 PY's per year. Remote data collection also means that hazardous areas can be closely monitored by frequent sampling to determine incipient movements. Another application of this technology could be as part of an early warning system to alert highway officials of a possible catastrophic failure.

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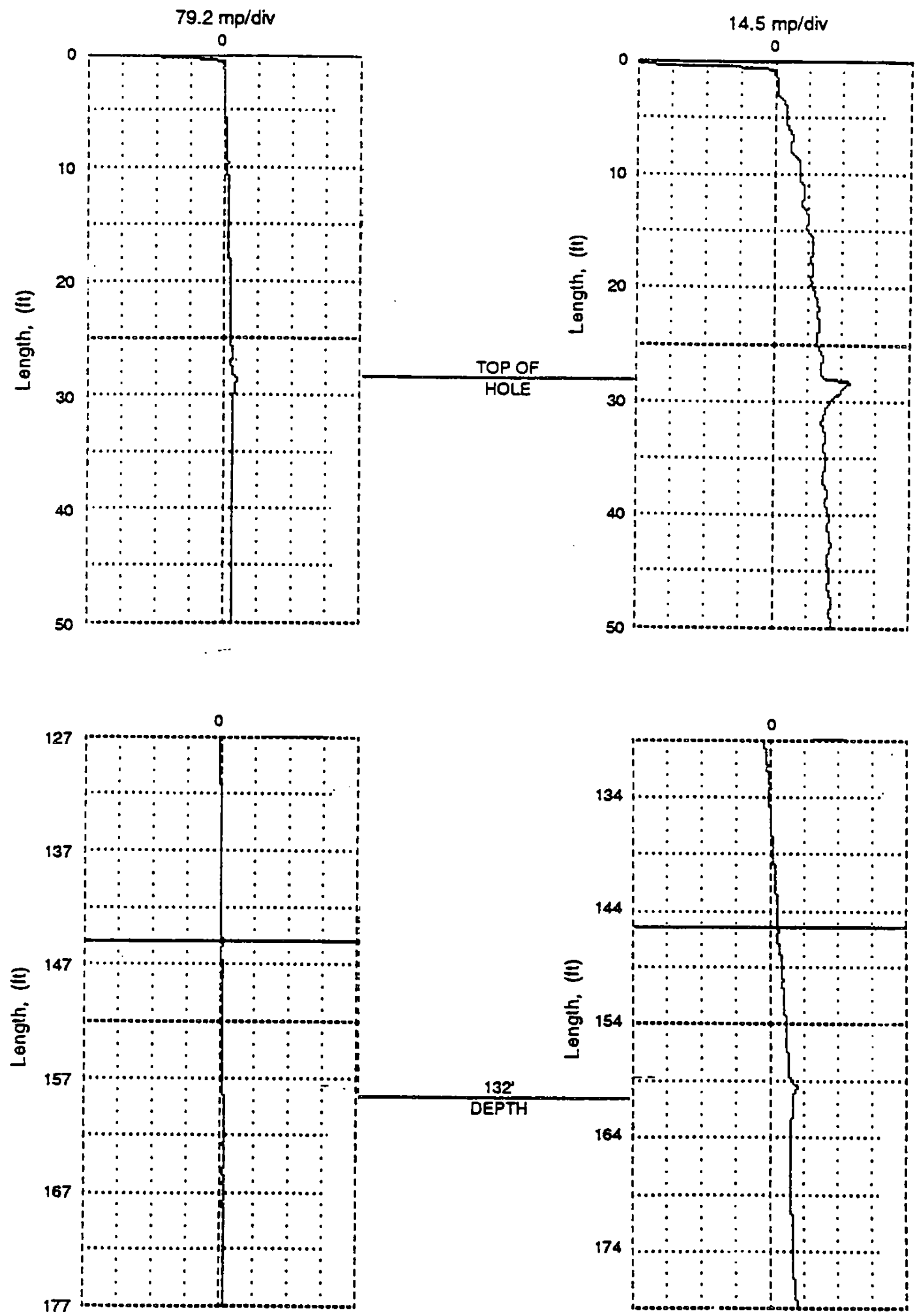

Figure 7 TDR output 148 days after installation. Note spikes where cable enters inclinometer hole and in vicinity of slide plane.

REFERENCES

CANN, L.R., AND STEINER, E.A. (1992). "Major Landslide Repairs in Southern California." In B.W. Pipkin and R.F. Proctor, Ed., *Engineering Geology Practice in Southern California*, Special Publication No. 4, Association of Engineering Geologists, Southern California Section, Star Publishing Company, Belmont, CA, 503-530.

DOWDING, C.H. AND HUANG, F.C. (1994). "Telemetric Monitoring for Early Detection of Rock Movement with Time Domain Reflectometry." accepted for publication, *Journal of Geotechnical Engineering*, ASCE.

DOWDING, C.H., SU, M.B., AND O'CONNOR, K. (1989). "Measurement of Rock Mass Deformation with Grouted Coaxial Antenna Cables." *Rock Mechanics and Rock Engineering* (22) 1-23.

FRANKLIN, J.A., AND DUSSEAULT, M.B. (1989). *Rock Engineering*. McGraw-Hill Publishing Company, New York.

HAMILTON, J.M., MALLER, A.V., AND PRINS, M.D. (1992). "Subsidence-induced Shear Failures Above Oil and Gas Reservoirs." In J.R. Tillerson and W.R. Wawersik, Ed. *Rock Mechanics. Proceedings of the 33rd U.S. Symposium*. Balkema, Rotterdam, 273-282.

O'CONNOR, K. M., AND WADE, L. V. (1994). "Applications of Time Domain Reflectometry in the Mining Industry." *Proceedings*, Symposium on Time Domain Reflectometry in Environmental, Infrastructure, and Mining Applications, Evanston, IL, <u>in press</u>.

THE ARIZONA INN LANDSLIDE
CURRY COUNTY, OREGON

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ABSTRACT

On Tuesday, March 23, 1993, a catastrophic landslide closed the Oregon Coast Highway, U.S. 101, approximately 11 miles south of the City of Port Orford. The highway closure occurred within a historically active area commonly referred to as the Arizona Inn Landslide, along the primary north to south route on the southern Oregon Coast. Approximately two weeks after the highway closure, a temporary alignment was opened to allow traffic through this section of highway.

The Arizona Inn Landslide is situated within rugged, mountainous terrain, immediately above and to the east of the Pacific Ocean. The area is commonly known for its landslide topography and numerous fault shear zones. Landslide movement of approximately 20 to 25 feet (horizontal and vertical measured at the highway), occurred following a 24 hour period where 7.33 inches of rainfall was measured. This rainfall event was preceded by a week where 2.25 inches of rainfall was measured daily.

Mapping and preliminary subsurface investigations indicated a slide volume of approximately 7 million cubic yards, a length of 1,900 feet, width of 700 feet, and depths of approximately 140 to 160 feet. Materials within the slide area are typically referred to as a "Melange" for their chaotically intermixed, jumbled, and sheared nature. Soil/rock units making up the slide mass consist of sandstone, siltstone, mudstone and altered metasediments which have been intensely sheared and deformed into a fault breccia gouge zone with clay.

Groundwater monitoring after the slide event revealed a confined aquifer within the slide zone (approximately 140 to 160 feet deep) and perched groundwater above the slide zone. Monitoring of the confined aquifer, following several weeks without rainfall, indicated piezometric heads between 8 and 30 feet above the ground surface. An automated data acquisition system (ADAS) has been installed at the landslide for continuous monitoring of groundwater, rainfall and slide movement.

In February of 1994, Squier Associates, a geotechnical consulting firm, was hired to continue the landslide investigation and develop potential mitigation measures.

INTRODUCTION

On Tuesday, March 23, 1993, a catastrophic landslide closed the Oregon Coast Highway, U.S. 101, approximately 11 miles south of the City of Port Orford (See Figures 1, 2, and 3). The highway closure occurred at a historically active area commonly referred to as the Arizona Inn Landslide, on the primary north to south route on the south Oregon Coast. Approximately two weeks later, the highway was reopened with a temporary alignment.

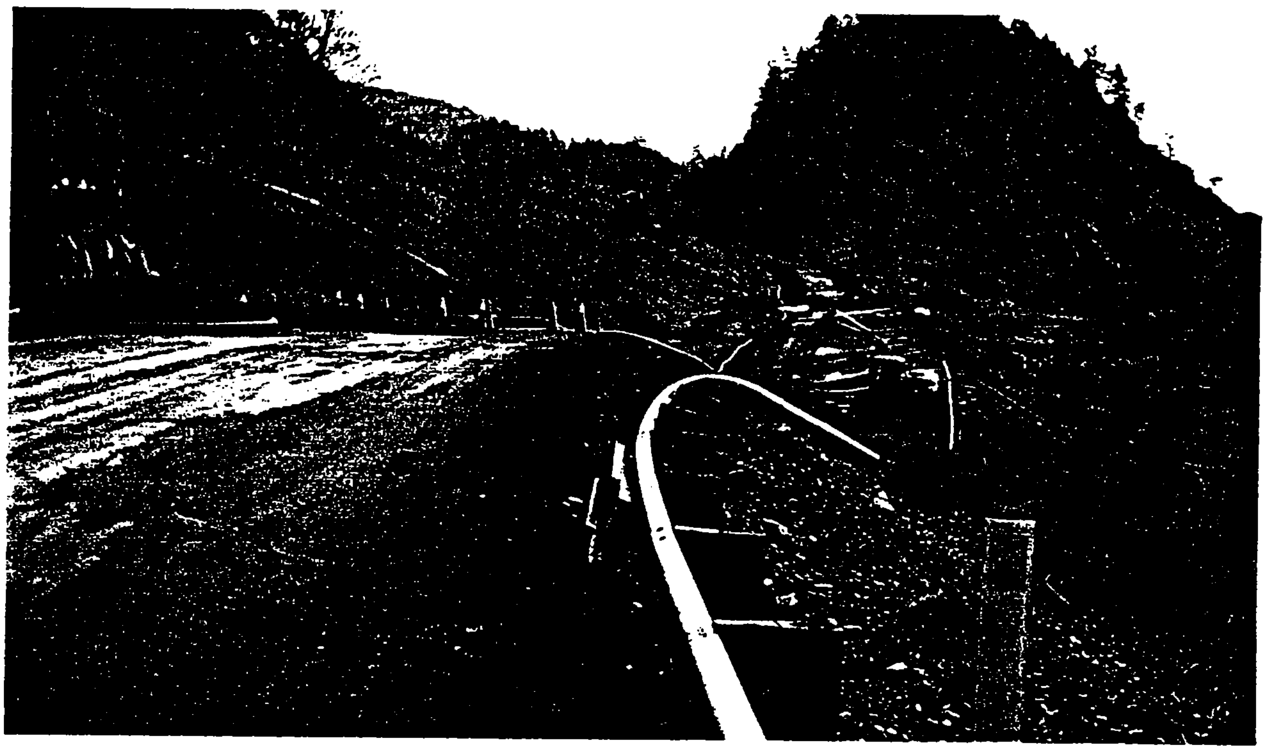

Figure 1. View of the highway two days after the landslide.

Immediately following the road closure, a geotechnical reconnaissance was implemented by the Region 3 Geology and the Geotechnical Engineering Groups. The reconnaissance was made to obtain a preliminary evaluation of the landslide conditions and to identify if feasible mitigation measures exist which will reduce the risk of a repeat movement event.

Figure 2. Oblique view of the landslide from the west.

BACKGROUND INFORMATION

General Site Conditions

The landslide is located immediately above and to the east of the Pacific Ocean within rugged terrain, steeply rising from the narrow beach area. Landslide activity within the area has produced the characteristic hummocky appearance of some of the upper slopes.

Localized ponding is evident on the mountain slopes to the northeast of the slide area and springs and creeks are visible throughout the surrounding hillsides.

The Oregon Coast Highway traverses the lower third of the landslide.

Climate and Vegetation

Rainfall in the region is heavy with an average of approximately 70 inches per year and a maximum recorded rainfall of 118 inches. The climate is moderate with an average winter temperature of about 45 degrees Fahrenheit and an average summer temperature of about 55 degrees Fahrenheit.

The general area is vegetated with timber and dense underbrush. Within the limits of the recent landslide, most of the larger vegetation has been removed.

Documented History of Slide Activity

The Oregon Coast Highway was originally constructed in the 1930's as a narrow two-lane highway. Its original location was approximately 50 to 100 feet up slope of its current location.

Since the initial construction of the highway, several landslide events have occurred at this location. Historically these major movements have occurred during or immediately following periods of high surf activity and above normal precipitation. Between these major landslide events, creep like movement has required regular highway pavement patching.

Listed below is a summary of the recorded history of the landslide since the original construction of the highway.

Year	Event
1938	In 1938, landslide movement was first recorded. The boundaries of the landslide at that time were similar to the limits recently recorded. Movement at the highway was approximately 25 feet, both horizontally and vertically. Five shafts, totaling 500 feet and a 40-foot long tunnel were excavated to evaluate the subsurface materials.
1954	Major landslide movement again occurred in 1954. The boundaries and magnitude of movement were similar to the 1938 movement. In an attempt to stabilize the landslide, approximately 500,000 cu. yds.

of material (slide debris) was excavated from the upper portions of the landslide and placed by conveyor between the current highway alignment and the ocean. At this time, the highway was also realigned to near its current location. By 1958 it was observed that approximately 20% of this relocated material had been eroded by the ocean.

Approximately 53 horizontal drains were installed, mostly on the bench, above the current alignment. These drains ranged from 10 to 159 feet in length. While some flowed full, many were dry.

1978	Slide movement was again observed. The magnitude of movement was similar to the 1938 and 1954 movements. Three slope inclinometers were installed to depths of up to 130 feet. Movement was observed at depths of 72, 122, and 80 feet.
Dec., 1981	Landslide movement again occurred. Movement similar to the previous events was observed. The roadway alignment was maintained by importing material and regrading through the period of movement.
	In July through October of 1982, construction consisting of clearing vegetation from the surface, surface grading, ditching, and culvert installation was accomplished within the slide boundaries. This work was done in an attempt to mitigate the slide movement.
Mar., 1993	Slide movement was observed on March 21, 1993 by the Port Orford Maintenance Crew. On Monday, March 22 and the morning of Tuesday, March 23, the slide movement accelerated. The Port Orford Crew worked 24 hours a day in an attempt to maintain traffic through the slide area by importing material and regrading the highway. By the evening of March 23, the rate of movement had accelerated to the point where the road could not be maintained. At this time, the highway was closed.
	During this time, a considerable amount of rain had fallen. On March 23, 7.33 inches of rain fell (measured at the Elk Creek Fish Hatchery). This followed a seven day period where approximately 2.25 inches of rain had fallen per day.
	By March 24th, the northerly edge of the slide at the highway had moved a total of 20 to 25 feet, horizontally and vertically. The southerly end of the slide had moved approximately 5 feet horizontally with little vertical displacement. Later, the southerly end of the slide moved an additional 5 feet, vertically and horizontally.

Surface monitoring and subsurface explorations began in order to determine the cause of movement, rates of movement and to evaluate preliminary mitigation measures.

April 8, 1993 — The highway was reopened to traffic. A temporary roadway consisting of a lightweight, sawdust fill had been constructed through the slide area. This alignment was constructed for use until a study for a permanent alignment and slide mitigation could be completed.

GEOLOGIC INVESTIGATION

General

Immediately following the movement on March 23, 1993, a preliminary surface reconnaissance and subsurface investigation was begun to evaluate the current conditions of the landslide and to determine what actions might be taken to safely reopen the highway to traffic.

The following is a summary of the general geologic history of the area, surface and subsurface investigation, and materials encountered during the investigation.

Regional Geologic Setting

Physiographically, the area is part of the Siskiyou Mountains of Oregon and is near the northern boundary of the Klamath Mountains Geologic Province. The rugged terrain rises dramatically from the surf, forming narrow beaches and steep cliffs. The landslide is located within a region which is commonly known for landslide topography (See the section of the DOGAMI Geologic Hazard Map, Figure 4). The geology of the area has also been described in DOGAMI Bulletin Nos. 69 and 90 and was the subject of a doctoral thesis by J.B. Koch of the University of Wisconsin in 1963.

The rock units that underlie the area are a heterogeneous assemblage of sandstone and mudstone which were laid on the Pacific Ocean floor during the Jurassic and Cretaceous Periods of the Mesozoic Era, approximately 100 to 200 million years ago. As the Pacific Plate slid beneath the North American Plate, the rock assemblages were scraped onto the North American Plate, causing the rock units to be severely deformed and altered. These units are so chaotically intermixed, jumbled and sheared that it is difficult to trace individual rock units for any distance. The term "Melange " (French for mixture) is well suited in describing the rock units of the area. In Northern California, the Franciscan Complex is similar and was formed during the same geologic time. Published geologic maps indicate the rock formations within the slide area are of the Humbug Mountain Conglomerate Formation, of early Cretaceous Age.

The area between Port Orford and Gold Beach contains three major shear zones which parallel each other with a northwest - southeast trend. These shear zones have a width ranging from 0.75 to 2.5 miles. Some geologists feel that the three shear zones are actually part of one larger shear zone, similar to the San Andreas Fault system in California. The rock units found adjacent to and within these shear zones are deformed, unstable, and so intensely altered by crushing that they are nearly unrecognizable. Landslides are common within the shear zones, particularly where they enter the sea and where the soft rock mass is subjected to wave erosion.

The rock units which make up the slide mass are sandstone, siltstone, mudstone and altered metasediments which have been intensely sheared and deformed into a fault breccia gouge zone with clay. Also included in these shear zones is serpentinite, a very soft, greasy, weak rock. Calcite veining and graphite is abundant in pieces of float found on the surface. Many rock surfaces have slickensides and a black shiny luster.

Surface Reconnaissance

The Arizona Inn Landslide is part of a much larger landslide mass which extends to the north and south of the recent movement. The active portion of the landslide was mapped and found to be approximately 1,900 feet in length from the head scarp to the beach area. The slide is approximately 700 feet wide and the maximum vertical relief from the beach to the head scarp is approximately 680 feet. A 20 to 25 foot vertical displacement was formed at the head scarp area because of the recent slide movement.

Because of the close proximity of the base of the slope to the ocean, the toe of the landslide is being eroded by wave action. The two prominent rock masses to the north and south of the Arizona Inn Landslide are large resistant blocks of rock incorporated in a softer, intensely deformed and sheared rock mass.

Surface exposures of rock and soil were generally obscured by a dense growth of grasses and brush. Bedrock exposures in the cliff above the beach were masked by the jumbled mass of soil and rock placed in 1954 in an effort to unload the head of the landslide and load the toe. Exposures were limited to the landslide scarps, access roads and drainage ditches. Mapping was difficult, however, a quartz mica schist (Colebrooke schist) body having slickensided surfaces and graphite coatings was located near the edge of the northern slide scarp. A coarse-grained sandstone outcrop occurs just above the schist body. These outcrops may not have any "roots" and are probably isolated rock masses within a gouge zone.

Several springs were observed, although it was difficult to ascertain whether the springs were caused by recent rain or if they flowed year round. Alders and other plants common to moist areas were used to help distinguish permanent wet areas during the mapping in May after the rain had abated. The locations of the springs and wet areas are shown on Figure 3.

Three monitoring lines were established across the landslide immediately after the main slide movement. These monitor lines were installed to determine the slide limits, rate and inclination of movement, and to later evaluate whether the highway could safely be opened. The monitor lines were surveyed on a daily basis for 2 weeks after the main movement occurred. The frequency of monitoring was reduced to a weekly basis when it was determined that little or no surface movement was occurring.

Subsurface Exploration

During March through May of 1993, a preliminary subsurface investigation program was completed by the Region 3 Geology Group. The subsurface investigation consisted of a total of 16 drill holes (See Figure 3 for the locations of the drill holes). Instrumentation consisting of open well piezometers and inclinometer casing was installed in the completed drill holes.

Later, an Automated Data Acquisition System (ADAS) was designed and installed for O.D.O.T. by Erik Mikkelsen of Landslide Technology Inc., to monitor of groundwater, rainfall and rates of landslide movement. The system consists of two programmable data loggers, vibrating wire pressure transducers, a biaxial electrolytic inclinometer sensor and a rain gauge. Data from this system is collected and transmitted to O.D.O.T.'s Roseburg office through the normal telephone system. The system can also be reprogrammed from the Roseburg office via the telephone system.

Drilling - Drilling commenced within 48 hours of the road closure in an effort to obtain a preliminary understanding of the slide mechanism. The first series of drill holes was concentrated immediately below the highway near the center of the slide mass. Later drill holes were made near the toe of the slide and above the highway.

Drilling was done by ODOT drill crews and two private contractors. All of the drill rigs were equipped with wireline systems, and mud rotary and coring methods were used. Drill hole depths varied between 50 and 250 feet. Drilling rates for the deeper slope inclinometer holes averaged 7 days per hole.

Drilling was extremely difficult and slow because of slide movement and the broken nature of the rock formation. Hard chunks of sandstone (2 to 4 feet in diameter) were encountered within a soft, crushed rock within a clay matrix. Water circulation losses were frequent and seizing or twisting off rods and casing was common after drilling through slide zones. A planned 350-foot hole was reduced to 240 feet due to difficult drilling conditions.

Several techniques were tried in an attempt to find a method which would increase the drill footage rate. Air rotary with soap or foam did not work, because the cuttings would not come to the surface. Coring techniques were used in SI 5-93 through the slide zone, from 130 to 250 feet deep, and took 14 days to complete. In an effort to expedite the installation of inclinometer casing and begin monitoring subsurface movement, non-coring techniques (mentioned below) were used to advance the holes.

The most efficient method was to advance the hole with a 3.5-inch diameter carbide insert drag bit on NMX size rods with a 4-inch I.D. casing. The smaller rod size allowed for cuttings to come to the surface and reduced the binding of rods. Binding was a problem when using rods with the same diameter as the bit. Four-inch ID casing was reamed using a casing advancer fitted with a tri-cone bit. Tri cone bits with carbide buttons were found to wear less than bits with teeth. While this system proved to be the best, it was still very difficult to advance the holes.

Inclinometer Installation - Plastic inclinometer casing (2.75-inch I.D.) with glued and riveted joints were installed with a cement/bentonite grout in six drill holes (SI 1-93 through SI 6-93). The slope inclinometer depths ranged from 195 to 245 feet. All shallow piezometer installations were also fitted with a "poor mans inclinometer" system consisting of a 2-foot long, 0.75-inch diameter iron rod. These rods were connected to a rope and lowered to the bottom of the casing. These rods are periodically raised to the surface to evaluate pinched zones which may indicate movement. No sheared or restricted zones in the piezometer casings have been noted.

Groundwater Monitoring - A total of 10 open well piezometers were installed at locations shown on Figure 3. The piezometers (1.5-inch ID PVC pipe) were installed to total depths between 50 and 100 feet with the lower 10 feet slotted. A sand pack was placed around the slotted section. Five-foot thick bentonite seals were place above the sand pack and at the hole collar.

Also identified during drilling was an artesian aquifer located approximately 160 feet below the ground surface. Piezometers P 7-93, P 8-93 and P 9-93 were installed in the aquifer and bentonite seals were placed immediately above the aquifer in an effort to assure that the piezometers are reading the aquifer head.

General Soil and Rock Units

The geologic investigation of the site failed to find evidence of the mapped Humbug Mountain Conglomerate. Instead, encountered was a “Melange” of rock which is typical of the Otter Point Formation of late Jurassic Age. The Otter Point Formation is a complex structural assemblage of highly varied rock units of diverse origin which is highly sheared and has a lack of structural continuity.

Listed below is a general description of the soil and rock units found during the subsurface investigation.

Residual Soil - The residual soil consists of loose to medium dense sandy silt with a trace clay, mottled brown to red, containing some siltstone and sandstone fragments. This unit is generally less than 5 to 6 feet thick since much of it has been removed during various surface drainage and slide repair programs through the years.

Rock Types - The predominant rock unit within the landslide is sandstone. Minor amounts of other rock types encountered include siltstone, mudstone, and altered metasediment. All of these materials are severely sheared forming a gouge material. The gouge material consists of crushed and ground up rock fragments which have degraded to form a soil matrix which accounted for 20 to 40% of the core samples. This soil gouge matrix consisted of clayey silt and silty clay with medium plasticity. Graphite coated fracture planes, secondary calcite veining and serpentinized rock were also present within the gouge.

Core recovery ranged from 60 to 80% and the average hardness for the sandstone fragments was R2. The softer mudstone and siltstone was found to have hardness range of R0 to R1. Most of the intact rock pieces were less than one foot in length and had slickensided surfaces.

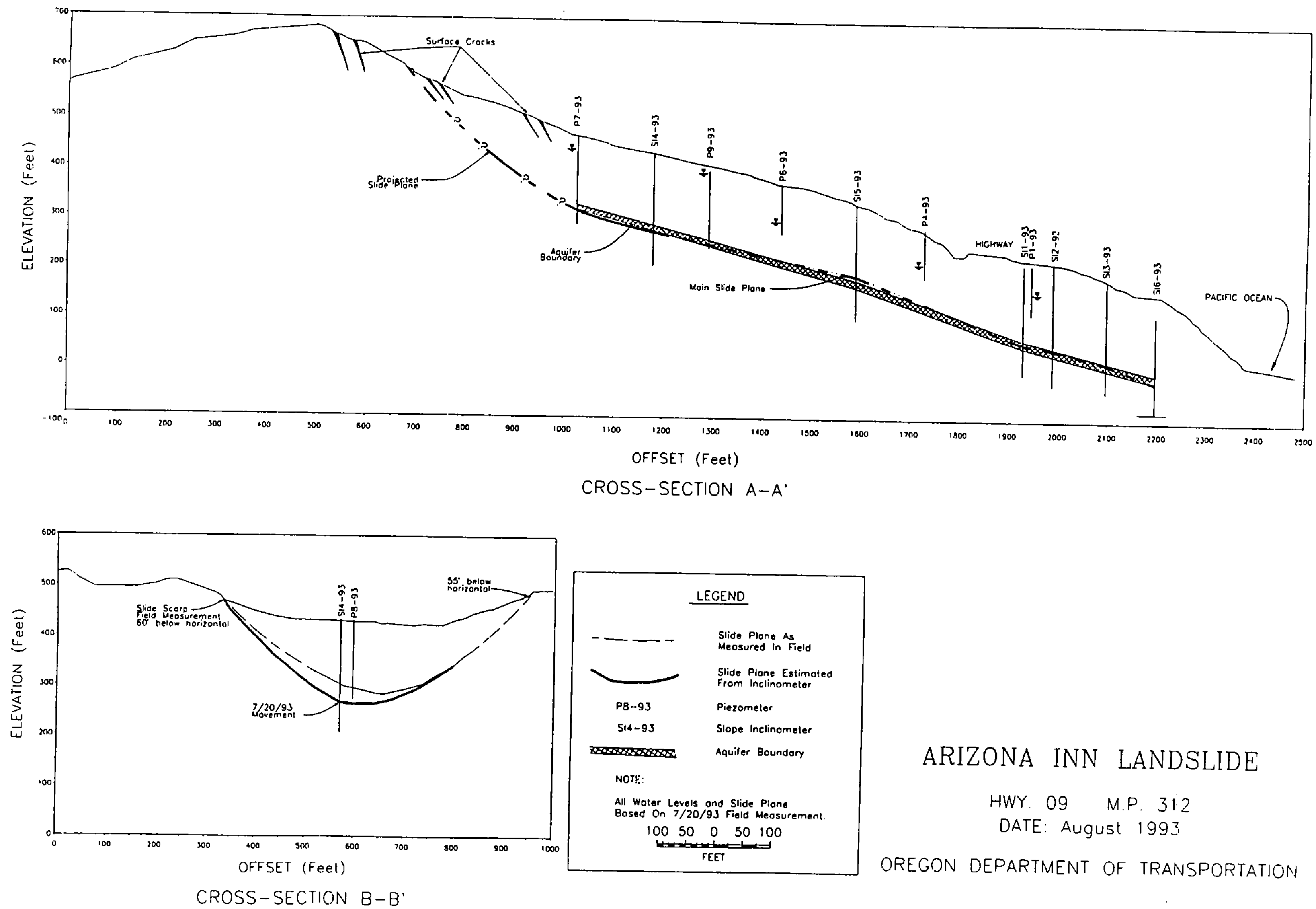

Figure 6. Cross sections of the landslide.

During the exploration phase a muddy/turbid area was frequently observed beyond the surf zone. State Maintenance personnel also mentioned that this muddy turbid phenomena was observed during the other major movement events. Local fishermen from Port Orford often have identified a localized low area in the ocean floor and a "fresh water boil" approximately a quarter of a mile off shore of the beach. It is possible that the muddy/turbid zone could be at or near the toe of the landslide and may also be a manifestation of the high pore water pressures exiting the slide zone.

Numerous grabens and scarps were mapped within the slide boundaries (See Figure 3). We have interpreted from the location of these grabens and scarps, that the slide moves in sections or has sections with differing rates of movement. This phenomena is probably due to variations in slide geometry or groundwater conditions.

Several areas of perched aquifers were found in the shallow piezometers. These aquifers are well above the slide zone and have caused of a number of seeps found in the existing highway cuts. These seeps and the material types encountered have caused the shallow slope failures within the larger slide mass.

Unfortunately, since the installation of the piezometers, limited periods of precipitation have been experienced in the area of the slide. Although groundwater readings during high periods rainfall have not been encountered, the artesian conditions within the slide zone have been monitored throughout the year (P 8-93 and P 9-93). Although these conditions exist, only creep like movement has been observed. While drilling inclinometer SI 4-93, a piezometric head of 7 feet above the ground surface was measured when the hole penetrated the aquifer. The drill rig pump used to pump water to the bit, at a pressure of 100 psi, was "upset" and 40 feet of drill rod was plugged with a coarse sand. This hole had to be abandoned and another hole drilled. Water continues to flow from the abandoned hole collar. At this time, we are unsure of the source of the aquifer, however, a possible source of the aquifer could be the neighboring shear zones. Because of these conditions, it is our interpretation that during extensive periods of rainfall, pore water pressures much higher than currently being measured exist within the slide zone. These extremely high pressures are the probable cause of the large slide movements.

FUTURE WORK

During this preliminary investigation, a study was made to evaluate the possibility of relocating the highway not affected by the landslide. Preliminary costs for relocating the highway were extremely high, in the neighborhood of 80 to 100 million dollars. This estimate did not include potential geotechnical mitigation's for geologic hazards such as soft foundation areas and landslides. The visual impacts of relocating the highway away from the ocean were also not addressed.

Because of the extremely high cost of the realignment option, a decision was made by O.D.O.T. to evaluate the possibility of reconstructing the highway through the landslide and developing a mitigation measure to eliminate or reduce the probability of future catastrophic movement.

Additional investigation and mitigation alternatives are currently being developed for O.D.O.T. by Squier Associates. This information will be discussed in a later paper titled "Engineering Geology and Hydrogeology Update, Arizona Inn Landslide" by G. Peterson, R. Squier, and D. Scofield.

BIBLIOGRAPHY

Beaulieu, John D. and Hughes, Paul W., 1976, Land use Geology of Western Curry County, Oregon, Oregon Department of Geology and Mineral Industries Bulletin # 90

Dott, R. H. Jr., 1971, Geology of the Southwestern Oregon Coast west of the 124th Meridian, Oregon Department of Geology and Mineral Industries Bulletin # 69.

Koch, John G., 1963, Late Mesozoic Oragenesis and Sedimentation, Klamath Province, Southwest Oregon Coast, University of Wisconsin Ph.D. Thesis.

Engineering Geology and Hydrogeology Update, Arizona Inn Landslide, Oregon

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Abstract

Catastrophic failure of the Arizona Inn Landslide, that occurred most recently on March 23, 1993, has been discussed by Oregon Department of Transportation (ODOT) (Kobernik, et. al., 1994). The landslide has a history of recurrent major movements, with persistent creep behaviour since highway construction in the 1930's. Major movement of the landslide is responsive to peak daily rainfall, following a period of significant rainfall. Major movement of the landslide disrupts and closes US Highway 101, causing severe economic hardship for the southern Oregon coastal communities. The on-going investigation by Squier Associates was initiated by ODOT to develop a suitable stabilization method for the active landslide mass. Final design of a selected stabilization alternative and an improved roadway alignment across the active landslide is planned for the upcoming months, leading to construction during the summer, 1995.

The landslide is located along the Oregon coast in the Klamath Mountains physiographic province, where numerous large and complex landslides exist. Bedrock in the landslide area is a heterogeneous complex melange assemblage consisting of sheared and chaotically intermixed sandstones and mudstones along with isolated higher grade metamorphic blocks. The landslide mass consists primarily of fine grained, intensely sheared, mylonitized sandstone and mudstone. It is bounded on the south by a very large rock block, with prominent rock headlands at the beach. A planar shear surface forms the north face of the rock mass, and appears to bound the landslide mass, both at the southern margin and in the subsurface. A larger landslide complex, that includes the active slide area called the Arizona Inn Landslide, extends to the north, where creep movement exists, but no catastrophic movements have occurred. The active portion of the landslide is approximately 700 feet wide and 1,900 feet in length from the head scarp to the beach. The toe of the landslide appears to be at the beach or within the surf zone. Uplifted wave cut terraces at the beach level indicate upward heave of the slide toe from recent movements.

Recent explorations (Spring 1994) and hydrogeologic investigations have further disclosed the landslide geometry, and have revealed localized, isolated and irregularly shaped zones of moderately permeable materials, with associated artesian pressures. However, much of the landslide and underlying melange is of low to very low permeability. The planar boundary of the south rock block, appears to provide a source of ascending ground water that is distinct from precipitation derived meteoric water. This deep source of ground water appears to provide a year round supply of confined ground water into the upper portion of the landslide, which is augmented by precipitation. Accelerated movements occur when the combined water sources produce critical hydrostatic pressures within the landslide.

Introduction

The Arizona Inn landslide is situated along the rugged Curry County coastline. Prominent rocky headlands and numerous landslide masses are present in the vicinity. This portion of the coast is composed of regionally significant tectonic melange equivalent to the Franciscan Formation in California. Consequently, the materials have been extensively sheared, intermixed and ground up. The resulting melange is a choatic assemblage of rock fragments, ranging from clay size to blocks hundreds or more feet in size. Within the landslide mass the melange primarily contains sheared Rocky Point Formation sandstone and mudstone. This block-in-matrix rock type has imprinted on it a mylonitic structural fabric, a foliation or alignment of the rock particles along shear planes resulting from extreme shearing and microbrecciation.

The Arizona Inn Landslide, exhibits creep-like movement during both wet and dry seasons, as do many landslides of this region. However, this mass also experiences infrequent large scale movements during critical rainfall events. Reported slope movements at the site date back to 1938, a few years after the original construction of the highway, and intermittent accelerated slide movements have occurred since. Large scale movements sever U.S. Highway 101, the Oregon Coast Highway, disrupting traffic, which has a considerable impact on the residents, visitors, and economy of coastal communities, both north and south of the landslide. Relocation of Highway 101 to avoid the landslide would cost many tens of millions of dollars, and would cross other landslide masses. Consequently, recommendations for a positive and cost effective method of stabilization are the focus of a phased geotechnical investigation by Squier Associates. The Site Plan, Figure 1, shows the boundaries of the landslide, and the location of borings, test wells, and instrumentation (both by ODOT and Squier Associates) and other relevant site features such as scarps, ground cracks, and springs.

Investigations and remedial work have been performed intermittently since the first major movements disrupted the newly constructed highway in 1938. Geotechnical investigations have included geologic mapping, exploratory drilling, and the installation of inclinometers. Remedial work has been accomplished, including unloading by excavation in the upper reaches of the landslide, and the installation of horizontal drains. Large scale movement occurred in December 1981, which led to regrading, surface drainage improvement, and roadway repair. However, none of these measures have been totally effective.

A critical rainfall event occurred in March 1993. A period of seven days of nearly continuous rainfall of approximately 2.25 inches per day preceded the failure. During this period accelerated landslide movement was observed, that resulted in cracking of the road pavement and required round-the-clock road maintenance. Following a one day lull in rainfall, a single day storm produced intense rainfall totalling 7.33 inches on March 23. The landslide movement accelerated rapidly during this storm resulting in 25 feet lateral and vertical displacement over a period of about 1 day. Rapid movements then slowed, and have returned to creep-like rates on the order of 1½ inches per year. Rainfall over the intervening period, to the present, has been significantly less, with no peak storm events.

The highly active portion of the slide, shown on the Site Plan, Figure 1, is over 700 feet wide at U.S. 101 and extends about 1900 feet upslope from the beach. Borings made in 1993 for the first time disclosed the basal zone of failure at a depth of about 150 to 170 feet, and located below sea level near the beach. The ground surface slopes at an average 19½ degrees. Inclinometers installed in 1993 have revealed a basal failure zone oriented at 12 degrees for the upper landslide area, and 19 degrees for the lower portion of the landslide.

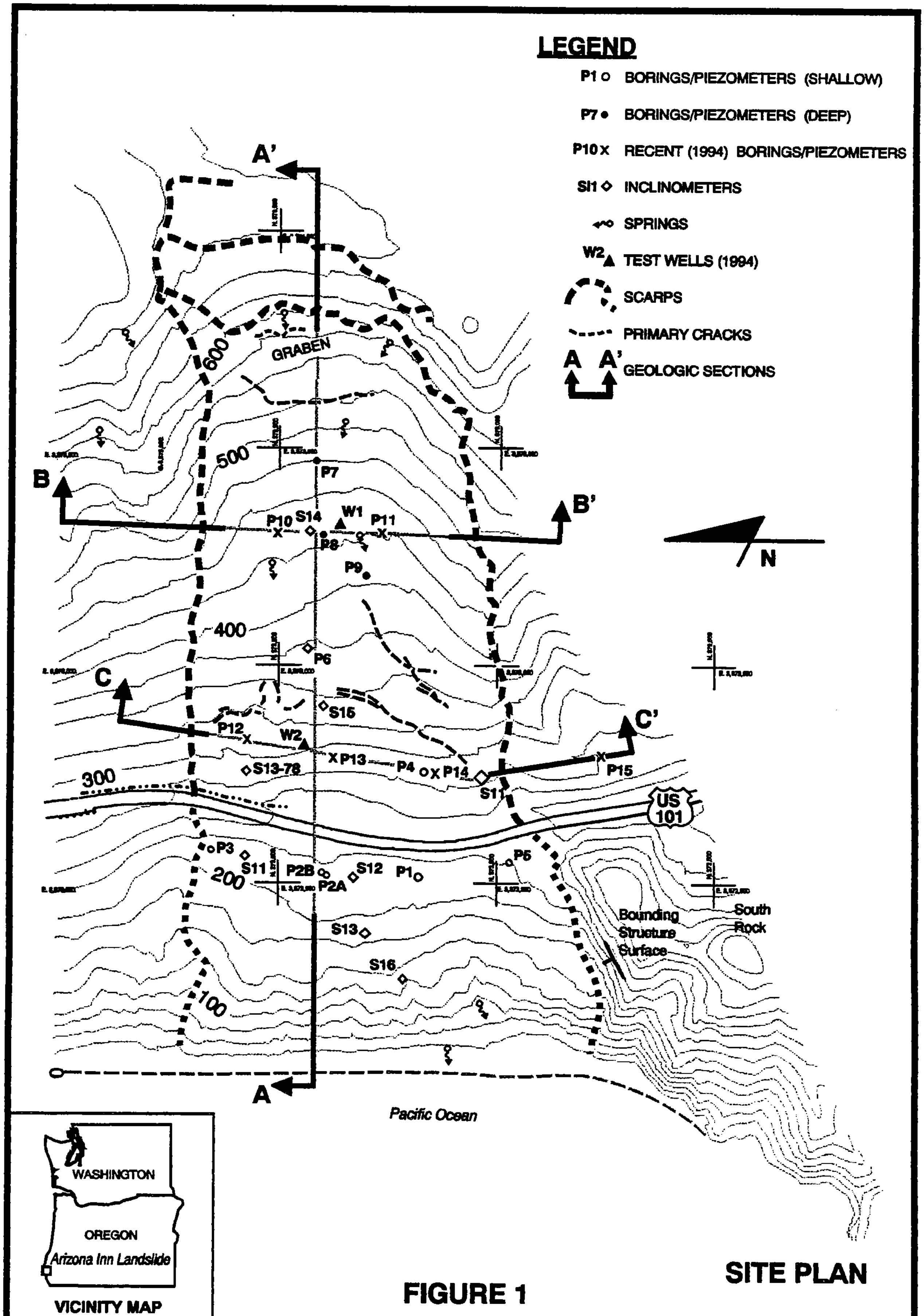
LEGEND
P1 ○ BORINGS/PIEZOMETERS (SHALLOW)
P7 ● BORINGS/PIEZOMETERS (DEEP)
P10 X RECENT (1994) BORINGS/PIEZOMETERS
SI1 ◇ INCLINOMETERS
SPRINGS
W2 ▲ TEST WELLS (1994)
SCARPS
PRIMARY CRACKS
A A' GEOLOGIC SECTIONS
A'
B
B'
C
C'
A
N
GRABEN
600
500
400
300
200
100
P7
P10
S14
W1
P11
P8
P9
P6
S15
P12
W2
P13
P4
P14
S1
S13-78
P15
US 101
P3
S11
P2B
P2A
S12
P1
P5
S13
S16
Bounding Structure Surface
South Rock
Pacific Ocean
WASHINGTON
OREGON
Arizona Inn Landslide
VICINITY MAP
SITE PLAN

FIGURE 1

Relatively high pressure artesian conditions were also disclosed in deep borings following the 1993 failure. Several open standpipe piezometers flowed at the ground surface for a number of months. Several have continued up to the present. Numerous springs exist, particularly in the upper reaches of the landslide and in the surrounding hillsides. Shallower piezometers revealed unconfined ground water levels. The geologic model developed as a result of the 1993 investigations suggested that a confined, pressurized aquifer exists at or near the basal zone of failure of the landslide. Consequently, accelerated movements during a critical rainfall event are attributed to increases in hydroststatic pressures along the zone of failure. High hydrostatic pressures were dissipated in some areas within weeks of the failure.

An evaluation of the geologic setting, depth of the failure plane, and apparent high ground water pressures have lead to the conclusion that only drainage alternatives would provide viable, cost effective long term stabilization. However, significant drawdown of water levels is necessary. Preliminary stability analyses suggested that overall reductions of 50 to 75 or more feet of piezometric head may be necessary. Current investigations and design work are focused on providing a cost effective, technically sound method of providing long term drainage. Principal methods of stabilization considered to date include:

1. Pump-out wells;
2. Long horizontal drains, with or without vertical drainage curtains;
3. Radial horizontal drains from shafts;
4. Radial drainage installed from drainage tunnel(s) below the slide mass; and
5. Vertical drainage installed to drainage tunnel(s) below the slide mass;

The current geotechnical explorations have focused on the following key issues: further definition of the extent and interconnection of the artesian zone; measurement of piezometric pressures in the landslide mass, in the artesian zone, and in the underlying bedrock; relationship of the unconfined water levels to the confined aquifer; relationship of piezometric pressures to precipitation; permeability and variability in the landslide mass and rock mass; source of the artesian pressures; and quantity of ground water.

Evaluation of these issues is being assisted by the drilling of additional exploratory borings, and the installation of multiple vibrating wire piezometers in each; continuous monitoring of piezometric pressures and rainfall, and evaluation of the data; and by stressing the ground water system using newly installed wells and aquifer pump tests. Continuous monitoring of one inclinometer, piezometric pressures and rainfall is being accomplished by an Automated Data Acquisition System (ADAS), which allows monitoring of 26 piezometers. Two aquifer pump tests have been conducted, one in the upper reaches of the landslide, the other in the lower reaches to characterize the aquifers, and hydraulic connections in the landslide mass, bedrock and lateral boundary conditions. As mentioned earlier, the locations of all borings and aquifer test wells are shown on Figure 1. A longitudinal geologic profile through the landslide, Cross Section A-A', is presented in Figure 2.

Site Description

A prominent rock ridge that forms the southern boundary of the landslide consists of steeply dipping, highly sheared and disrupted beds of sandstone and mudstone with shaly interbeds correlative to the Rocky Point Formation. South Rock, a prominent headland extending into the surf, provides good exposures of the sheared sediments comprising the southern rock ridge, and

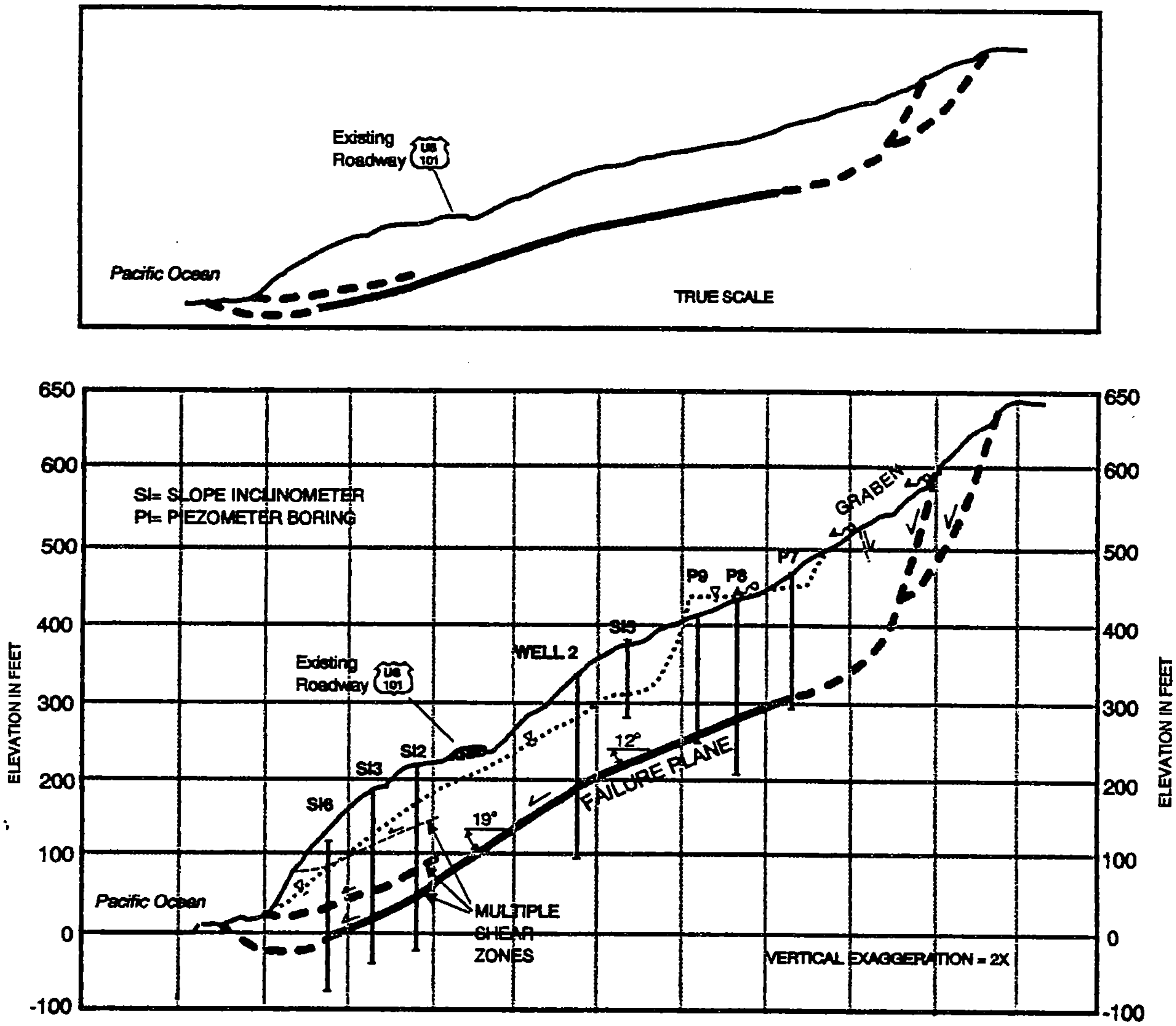

Figure 2 Cross Section A-A'

other structural features of interest. Abundant calcite and trace pyrite mineralization on joint surfaces is present. A gouge-filled shear zone of substantial thickness was disclosed near the bottom of boring P15, in the rock ridge. These same bedrock source materials have been encountered throughout and beneath the landslide mass. Within the landslide area, however, the material is more extensively sheared, and is reduced to a fine grained sand, silt and clay matrix with typically a relatively small proportion of gravel to cobble size sandstone and mudstone rock fragments.

Although we estimate the size of the South Rock block to be at least 800 feet wide (north to south) by 1800 feet long (east-west), it is probable that this too, represents an isolated block in the melange. A prominent planar facet on the north side of the rock block exposed between the highway and the beach appears to represent an extensive planar boundary of the rock mass.

This face has been mapped as a fault, and has that appearance at the ground surface and in aerial photographs. The planar block boundary is oriented at about N61°E, 60N dip. Subsurface projection of this structural boundary has been accomplished, and appears verified by subsurface data. This planar bounding surface of the rock ridge appears to form the southern boundary of the landslide.

The northern bounding lateral scarp of the active landslide mass, particularly in the upslope portion of the landslide, represents a transition in material types. North of the lateral scarp, large exposed rock blocks (with dimensions on the order of hundreds of feet) are present, and include metamorphics (schist and phyllite) and conglomerates. Source formations for these blocks are most likely the Colebrooke Schist and Humbug Mountain Conglomerate. Also, boulders exposed along the beach north of the landslide display a variety of metamorphic rock types, including schist, phyllite, greenstone and talc. A somewhat larger proportion of boulders appears present on beach to the north, as compared to the landslide mass.

The upper portions of the landslide contain a graben, shown on Figure 1, with prominent head scarps, and a retrogressive smaller scarp near the ridge crest. Other smaller scarps are found crossing the ridge crest on the norther portion of the landslide. Portions of the upper head scarps have failed as earths flows, and display intermittent spring activity.

Uncertainty has existed with respect to the location of the landslide toe. Inclinometer data reveal the elevation of shearing to extend down to -44 feet, mean sea level, some 300 feet inland from the beach. Most shear motion, however, is measured at and above elevation -23 feet, and multiple shear planes are present as shown on Figure 2. Reconnaissance at the beach reveals elevated wave cut terraces, minor spring activity, and clayey melange materials underlying thin beach sand deposits. These features support the interpretation of a passive wedge being uplifted at the beach, and are consistent with stability analyses of the landslide mass. Elevated wave cut terraces with preserved recent beach deposits mantling their surface are observed, particularly near the southern landslide margin where about 4 or more feet of uplift is apparent.

Site Hydrology

The area receives 75 to 80 inches of precipitation a year. Most of this falls as rain during the winter months between October through April. However, a review of the last twenty two years of precipitation records for the Port Orford 5E station, located about 10 miles north of the site, shows that 46 percent of the total precipitation falls during storms that produce two or more inches per day. The amount of recharge that occurs into the landslide mass during heavy rainfall events is uncertain. However, infiltration is probably relatively high due to the predominant grass cover and numerous open cracks related to movements.

Spring activity on the landslide is known to occur nearly up to the ridge top at elevation 600 feet, and outside the landslide to nearly 700 feet elevation. Conductivity measurements of the water suggest that most of the springs may be from precipitation. However, one spring and at least two borings produce sulfurous, poor quality, water that has a higher pH and is slightly more conductive. For example, well W-1 and boring P-11, located on Cross Section B-B', produce sulfurous water. A sulfurous spring is also located near elevation 75 feet near the southern landslide boundary. This sulfurous water is thought to be water that has followed a deeper, and thus longer percolation path than the direct precipitation and local recharge for the other springs. Trace pyrite material in the South Rock block indicates the presence of sulfides that would be leachable to percolating waters.

The historic rainfall records for Gold Beach and Port Orford have been reviewed. The cumulative monthly precipitation record shows that major failure of the landslide does not correlate with the long term climatic variation. This is suggestive that the triggering event for the rapid acceleration and catastrophic failures of the landslide is not related to ground water recharge from distant recharge sources. Review of seasonally cumulative rainfall during the winters has been accomplished for 71 winters since the 1890's. Five catastrophic failure years do not reflect high seasonal precipitation. Hence, the slide is not particularly sensitive to wet winters. Correlations for two month, one month and single day rainfall were also developed. The best correlation rainfall to rapid failure is obtained with one day maximum precipitation shown on Figure 3. Of the three highest one day storm events since the 1981, the slide failed on two of them (December 1981 and March 23, 1993). The March 23, 1993 storm was preceded by seven days of approximately 2.25 inches per day rainfall. The storm itself dropped 7.33 inches in 24 hours. The December 1981 peak daily rainfall was 9.40 inches. This storm was preceded by four days that dropped 5.50 inches. However, no major movements were recorded during or following the December 3, 1987 storm. This storm occurred five years after extensive regrading and surface drainage improvements were completed on the landslide following the 1981 failure. The peak 24-hour precipitation was 7.75 inches. The preceding nine days were relatively dry. However, the succeeding three days were wet with nearly 8 inches of rain. Absence of a significant period of rainfall before the peak rainfall event may explain why this event did not result in accelerated movements.

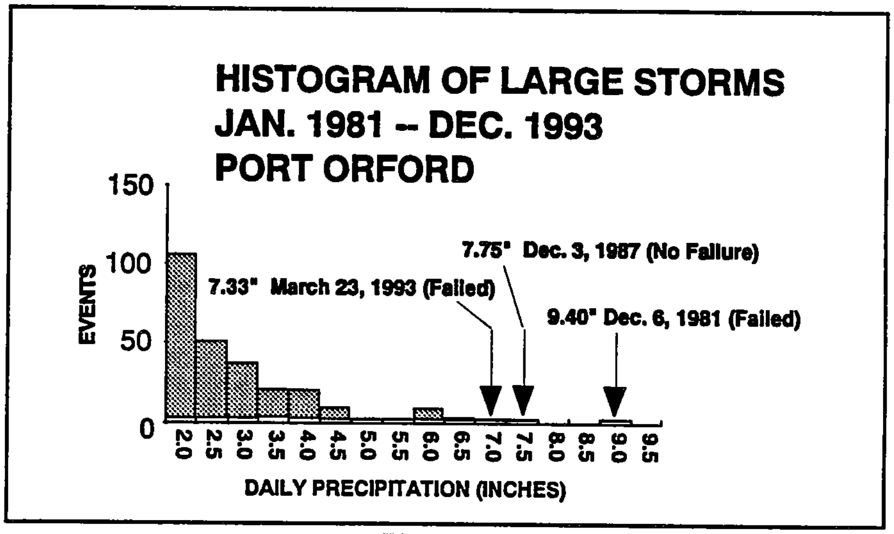

Figure 3

Borings

To supplement and expand upon prior explorations by ODOT, a program of six borings and two water wells was accomplished during the Spring of 1994. The borings and wells were located in two areas transverse to the central axis of the landslide, and were planned to obtain samples, and to develop geologic and hydrogeologic data for the landslide mass. Geologic Cross Sections made on Sections B-B' and C-C' (refer, Figure 1) are presented in Figures 4 and 5. The borings were extended far enough below the assumed basal failure plane to allow design information to be gathered in consideration of a deep drainage tunnel. Depths of the borings ranged from 190 to 225 feet. One boring was located outside the landslide mass in the bounding South Rock ridge to evaluate rock mass properties, considering that a drainage tunnel approach from the rock ridge may be a viable drainage alternative. Total drilling footage of 1259 feet was completed in April and May, 1994. To develop important hydrogeologic data from the borings, borehole packer tests were performed, and multiple pressure transducer piezometers were installed. Borings were accomplished by Longyear Company of Spokane, under contract to Fujitani Hilts and Associates.

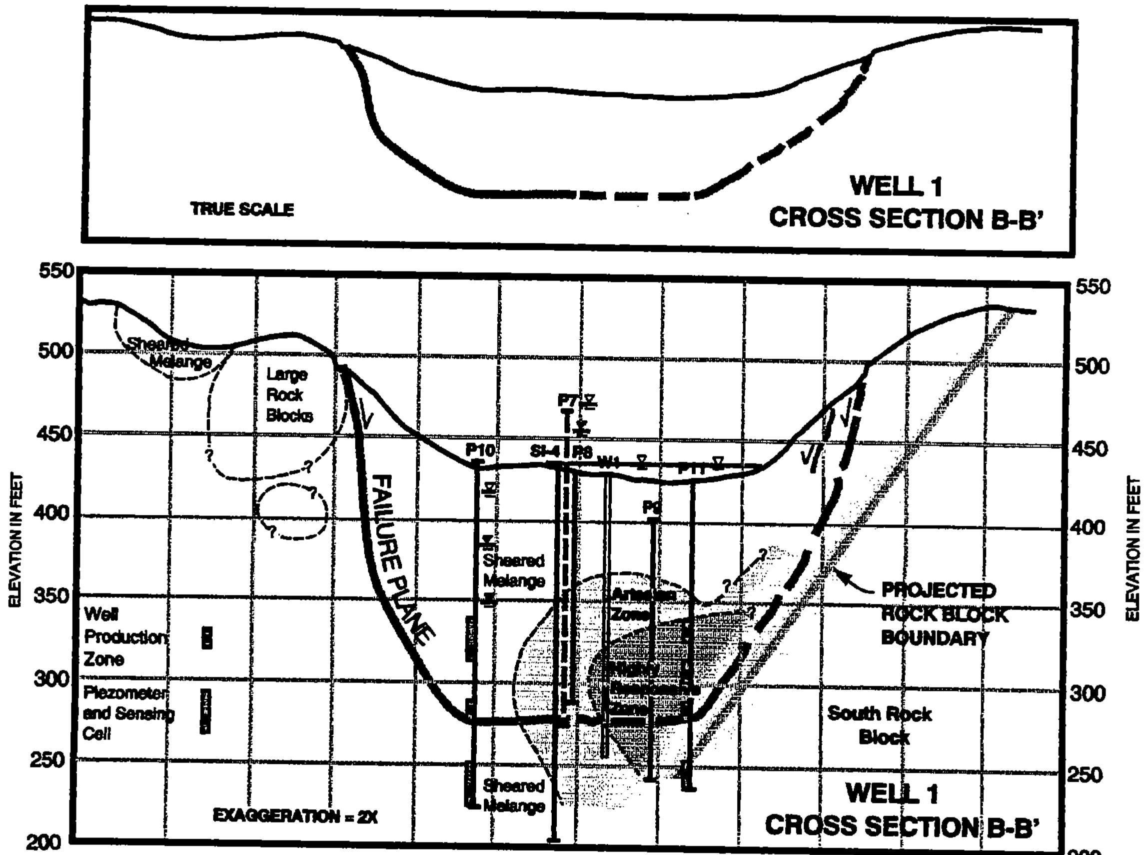

Figure 4

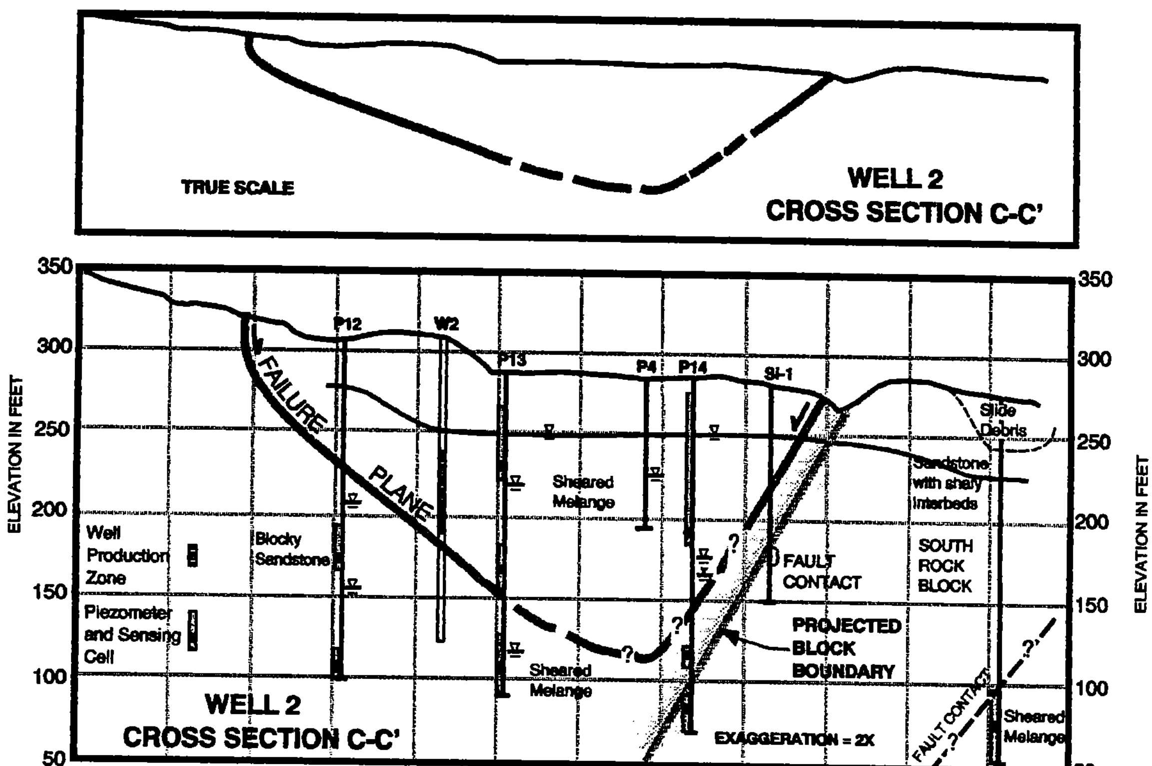

Figure 5

Because prior explorations had disclosed that distinct stratigraphic units were not distinguishable in the core, and because of the significant depth to the failure plane, the upper portions of most borings were blank drilled using a casing advancer system with mud. At a selected depth, HQ wireline coring using a triple wall tube system was begun. External casing was advanced and maintained relatively close to the coring bit. Use of drilling mud was minimized, and altogether excluded in the piezometer and packer test locations, thereby allowing confidence in hydrogeologic data obtained. Poor borehole stability in certain areas provided drilling and in situ testing challenges, but a high degree of success was accomplished in the packer tests and pressure transducer installations.

Instrumentation

Vibrating wire pressure transducers were installed at selected loctations to allow long term monitoring of water pressures. The sensing zone for piezometer installations was constructed of coarse sand backfill; seals between sensing zones consisted of chipped bentonite. Although time consuming in installation, successful installations of up to four piezometers per boring (including an upper PVC standpipe) were accomplished.

To allow continuous monitoring, the automated data acquisition system (ADAS) installed initially by ODOT was significantly expanded. All newly installed pressure transducers, and several ODOT installed transducers were connected to the datalogger. In total, some 26 piezometers are automatically monitored on the site, with remote data collection and programming possible through phone / modem connections. Programming of the ADAS systems allows user selection of hourly data, daily maximum and minimum values, or a test mode which provides user configurable reading increments down to ½ minute. The ADAS system was employed during pump tests to assure coverage of piezometric pressures at a high reading frequency.

Water Pressure Tests

To evaluate field permeability of the materials comprising the landslide and underlying rock mass, water pressure tests were conducted using a downhole packer system. A single wireline packer, with integral pressure transducer in the test zone, was utilized to test selected zones as the boring proceeded. A total of 435 linear feet of boring was tested in a total of 23 test zones. In addition, five tests could not be successfully completed due to inability to successfully seat the packer. The results of the packer tests are presented on Figure 6. The tests revealed that most of the material has low to very low hydraulic conductivity, i.e. less than $1X10^{-4}$ feet per minute. A few zones, located generally near the failure zone in the upper tier of borings revealed moderate hydraulic conductivities, on the order of 10^{-3} feet per minute.

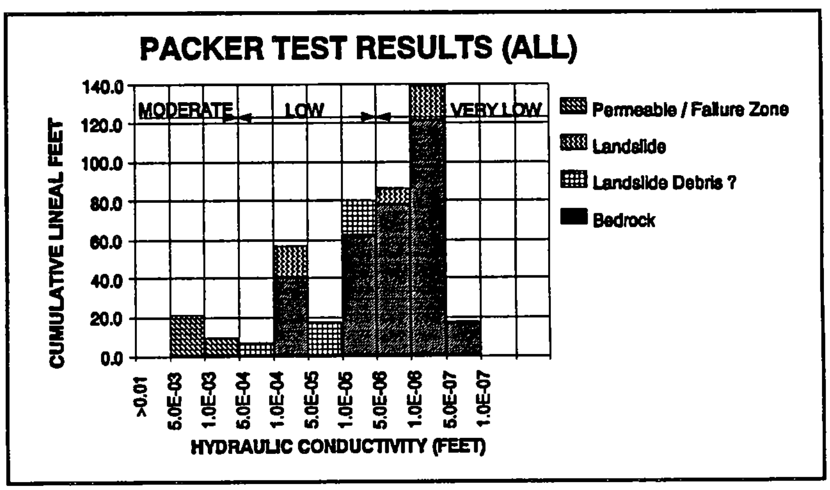

Figure 6

Aquifer Pump Tests

Two water wells were installed to allow aquifer pump tests for an evaluation of field permeability, transmissivity, connectivity, and storativity of the confined aquifer and landslide mass. In addition, the wells were intended to test the viability of a pump-out well stabilization scheme. Well drilling was specified to use cable tool drilling techniques to assure completion of the wells to the desired depths, and in consideration of earlier difficulties with different techniques and equipment. The wells consisted of 8-inch diameter cased holes: one was completed with stainless steel well screens, and another with perforated casing and a PVC liner pipe. Bandon Well and Pump Company was subcontracted to construct the wells.

Two 72 hour aquifer tests were conducted at the installed wells following completion of the exploration program. Drawdown achieved for the pump tests was about 110 feet and 60 feet for wells W-1 and W-2, respectively, as shown on Figure 7. The two pump tests provided distinctly differing results that indicate the range and variability of hydrogeologic conditions in different segments of the landslide.

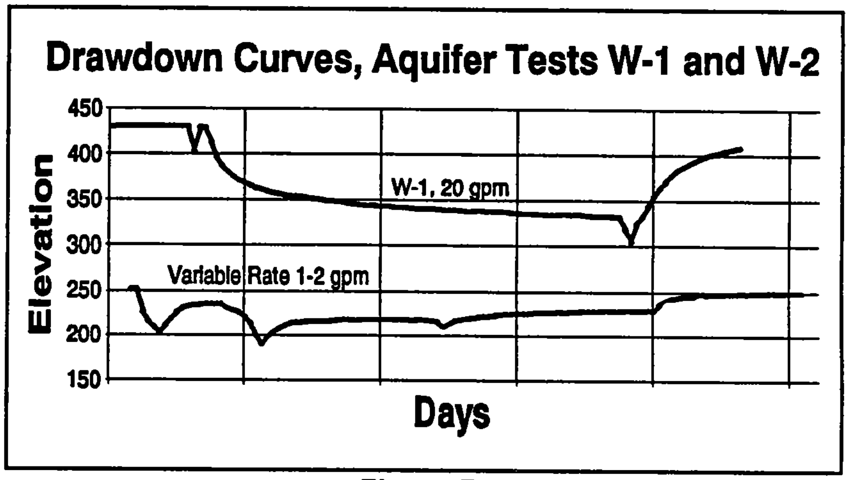

Figure 7

Well W-1, located in the upper tier of borings, was tested at an average rate of 20 gallons per minute over 72 hours. At the end of this period, a flow rate of 65 gpm was maintained for about 1 hour to further stress the aquifer. Drawdown during test in Well No. 1 extended to the south and southwest of the well. The amount of drawdown in the observation piezometers was related to the direction and the vertical distance from the inferred South Rock ridge bounding structure and not to the radial distance from the pumping well. In this regard, well W-1 communicated at nearly 100% efficiency with borings that encountered the projected boundary of the south rock block, i.e. borings P-11 and P-9. However, the pump test did not communicate efficiently (28%) with the closest observation well (P-8), located only 80 feet away. Virtually no influence was seen on upslope piezometers, nor on springs immediately adjacent to the well site. The pump test revealed the moderate to high interconnectivity within a ground water cell that is inferred to be somewhat controlled by the presence of the south rock block boundary, as shown on Figure 4. As previously mentioned, well W-1 produced distinctly sulfurous, poor quality water, as did the hydraulically connected boring P-11. No other borings have disclosed sulfurous water. Origin of the sulfurous water is likely through a plumbing system associated with the south rock block or its planar northern face. Data to the present suggest that the artesian head in this area is likely constant, or nearly so, on a year round basis. The location of the sulfurous spring at elevation 75 feet, also suggests a relationship to the structural boundary of the south rock block.

Well W-2 aquifer test demonstrated, by contrast, the very low permeability of the mid-section of the landslide. During drilling of the well, very low permeabilities were indicated. Completion of the well in the landslide debris, as shown on Cross Section C-C', Figure 5, tapped the most productive portion of the well, yet was not able to maintain a 2 gpm flow rate. Essentially no response was observed in nearby piezometers to the maintained drawdown accomplished during the test. Based on the water pressure tests, measured water levels, and the aquifer test, this central portion of the landslide is inferred to form a hydraulic barrier, impounding ground water and restricting seepage from the upper portion of the landslide.

Because of the limited number and distribution of piezometers, only generalized ground water contours can be developed. Ground water levels are shown on the geologic Cross Sections Figures 2, 4 and 5. The variation in hydraulic gradients along the slide profile and across the slide suggest that the slide mass is highly heterogeneous with respect to hydraulic conductivity and may be divided into isolated hydraulic cells.

Preliminary Conclusions and Design Alternatives

Explorations to date at the Arizona Inn Landslide have disclosed that the landslide is comprised of extensively sheared sandstone and mudstones that form a fine grained melange, with a foliated, mylonitized fabric. Variable amounts and distributions of relict angular rock fragments exist, but within the landslide most rock fragments are cobble to gravel size, with few intact boulders. On the south and north, larger rock fragment sizes exist. On the south, the South Rock ridge bounds the landslide. A planar bounding surface of South Rock is a key geologic and hydrologic component of the landslide.

The landslide exhibits heterogeneous permeability, but generally is comprised of low permeability materials. Localized moderately permeable zones exist, and are generally associated with the failure surface of the landslide. The central portion of the landslide is generally of low permeability, and uncertainty exists regarding permeability of the lower portion of the landslide mass. In the upper reaches, the moderately permeable zones are effectively isolated, one from another, and from lower portions of the landslide.

Based on the geologic and hydrogeologic investigations, the landslide appears to contain isolated confined and unconfined aquifers with especially high hydrostatic pressures in the upper portion of the landslide. Two distinct ground water sources have been identified: meteoric water and deeply circulated, sulfurous water. Sulfurous water appears to ascend into the slide mass along flow paths associated with the planar boundary of South Rock ridge. Isolated, cellular aquifers in the upper slide mass probably exhibit high head year round, and are most likely related to deeply circulated ground water.

The mode of failure inferred for major, catastrophic movements of the landslide consists of the following. During periods with significant rainfall, infiltration of meteoric water occurs, which combines with, and builds upon, the existing high hydrostatic pressures. Together, critical piezometric pressures are reached, which initiate major movements of the landslide. Transmission of the peak hydrostatic head to lower portions of the landslide likely occurs, possibly along the south rock block or within other zones. Acceleration of the landslide mass occurs in response to the peak pressures in the upper slide mass, effectively driving the middle and lower portions of the slide downslope. Hence, the trigger for the rapid catastrophic failures appears to be correlated with critical characteristics including: a prolonged period of significant rainfall, leading to a high intensity, short duration peak rainfall period.

Consideration of design alternatives to provide drainage has progressed to the conceptual level. No final design approach has been selected. Only a drainage or ground water control approach appears practical to control future slide movements. At this time, two primary alternatives appear feasible for long term stabilization. Fundamental requirements of the two primary alternatives include the ability to lower and maintain piezometric heads below critical levels in the landslide mass. The ability to accomodate intense storm events, with rainfall totalling 16 inches or more in a week, needs to be considered in the design.

One drainage concept being considered would be an extensive network of horizontal drains. A conventional approach would consist of surface drilled, relatively long (700 feet or more), drain holes at various elevations, and at various horizontal and vertical orientations. A variation of this alternative would be the installation of multiple levels of somewhat shorter length horizontal drains from shafts constructed within the landslide mass. Another concept is the construction of a drainage tunnel beneath the landslide, with radial drains extending up and into the landslide mass. Alternative portal or shaft access sites are under consideration for the tunnel. An evaluation of tunnelling conditions and geometric requirements suggests that a drainage tunnel is viable, from a constructibility perspective. Support requirements would range from rock dowels and shotcrete, to lattice girders, mesh and shotcrete. Discharge outlet(s) from the tunnel would daylight outside the landslide mass, allowing full gravity drainage.

These conceptual drainage schemes are presently under review and consideration by the Oregon Department of Transportation, among others. Major considerations in the selection of a design choice are:

1. Cost vs. benefit, i.e. effectiveness;
2. Longevity, i.e. expected design life;
3. Maintenance costs;
4. Degree of stabilization, i.e. design factor of safety; and
5. Assessment of risk, including seismic risk.

References

Kobernik, R.M., Toor, F.N. and Watanabe, R.F., Arizona Inn Landslide, Curry County, Oregon, 1994. National Highway Geology Symposium, Portland, Oregon

Evaluation of Indiana Aggregates for Use in Bituminous Highway Overlays

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ABSTRACT

Bituminous overlays are placed over concrete pavements which have undergone years of wear from highway traffic. A smooth riding surface results and the friction level of the pavement becomes more critical. The coarse aggregate in the bituminous overlay must supply the primary roughness to yield needed resistance for braking.

Polishing of the coarse aggregate commonly occurs in bituminous pavements, which greatly reduces the frictional resistance. In Indiana, dolomites have been found to provide better frictional resistance on average than do limestone aggregates. In this study the physical characteristics of the aggregate are being compared to polish resistance and surface friction properties. A detailed petrographic examination is also included. Polish susceptibility is determined using the British Wheel and Pendulum testing equipment. A BPN, British Pendulum Number, is obtained.

Under current highway specifications, dolomites must contain a minimum of 10.3% elemental magnesium to qualify as an acceptable aggregate for bituminous overlays. Dolomites with varying magnesium contents will be tested using the British Wheel and Pendulum method. Acid-insoluble content of these rocks have been determined. Such residues consist primarily of quartz and clay. It is expected that the quartz will provide increased frictional resistance whereas clay will yield a reduction. Thin sections of aggregates are being analyzed to determine mineral composition and texture.

Crushed river gravels are another aggregate source of interest for bituminous overlays. Their performance will be evaluated using the British Wheel and Pendulum test. A petrographic evaluation will be made, examining details of rock type and texture. The purpose of the research is to determine how well aggregates of different composition and texture resist polishing and to correlate aggregate properties with their frictional resistance.

INTRODUCTION

An acceptable dolomite aggregate for bituminous surface overlays in the State of Indiana must contain a minimum of 10.3% elemental magnesium. Historically, dolomite aggregate has been specified for use primarily on the basis of field trials and past performance. Other types and sources of aggregate have also been available for overlay purposes. Due to the lack of field experience or a proper procedure that identifies limiting characteristics, the alternative sources have rarely been used. The fundamental goal of this study is to determine the petrographic and physical characteristics that constitute a good polish and high frictional resistant aggregate and to develop procedures which would allow expeditious evaluation of aggregate sources.

The aggregate study is scheduled to extend for a period of approximately two years and has recently entered the second year of evaluation. The experimental design includes the following:

a. Sampling
b. Laboratory Testing
c. Field Friction Testing
d. Test Section Inspection and Sampling
e. Data Analysis
f. Draft Specifications and Report Preparation
g. Report Review

This program is designed to approach the evaluation of aggregate sources from two perspectives. The first involves collecting crushed aggregate samples from quarries throughout and bordering the State of Indiana. Collected aggregate is then evaluated on the basis of laboratory tests consisting of sieve analysis; a detailed petrographic examination to include average grain size, grain shape, percentage of groundmass, void content and composition; and a review of historical data concerning physical properties such as sodium sulfate soundness, freeze-thaw, specific gravity and absorption. Other tests sensitive to aggregate type include acid-insoluble residue on the carbonate aggregate and magnesium content on the dolomite. All aggregate sources will be tested for polish and friction properties. Evaluation of these two properties is being determined through the use of (1) the British Wheel which polishes the aggregate and (2) the British Pendulum which measures the friction values before and after polishing.

The second approach has been termed Field Friction Testing and Test Section Inspection and Sampling. BPNs, British Pendulum Numbers, and AFNs, Average Friction Numbers, are correlated to determine the relationship between laboratory and field values. This method entails measuring the level of friction from a selected section of highway through the use of an instrument called the towed friction trailer. After the AFN has been determined for a sample section of highway, the bituminous overlay in the area of trailer wheel contact is cored. The friction level at the top of the cored samples is measured using the British Pendulum. Afterwards, the cores are stripped of bitumen, and the aggregate is molded into coupons and tested in the same manner as in the first approach.

SAMPLING

Dolomite is the most common aggregate used in bituminous surfaces in the State of Indiana. From that perspective, nineteen sources of dolomite have been sampled for a comparative study. One of the proven best performers has been selected as the reference aggregate from which all other samples will be compared. Other aggregate types included in this study are six river gravel sources, two limestone sources, and two sandstone sources.

The coarse aggregate gradation currently used in Indiana highway overlays is the Indiana #11s. This raw aggregate has been crushed so that 100% passes the 1/2 inch sieve. Details of the gradation are as follows:

Sieve size	Percent Passing
1/2"	100
3/8"	75-95
#4	10-30
#8	0-10

Stock piles were sampled using ASTM procedure D 75, Standard Practice for Sampling Aggregates. When possible, power equipment was used to collect and overturn the aggregate to reduce the possibility of biased results. Where power equipment was not available, stockpiles were sampled in three increments, from top to bottom. Approximately 60 kilograms were collected from each site.

LABORATORY TESTING

Sieve Analysis - The type and quantity of material in specific gradations are expected to play an important role in polish and resulting friction values. The coarse material in a given aggregate source is considered to be responsible for friction properties. An abundance of fine material may result in lower friction values. For most gravel sources, a large portion of the fraction greater than the #4 sieve consists of carbonates with the remainder primarily igneous and metamorphic. The presence of these harder constituents may show that gravel sources are superior to dolomites.

Aggregate samples were reduced to a representative size of approximately 2.5 kilograms using ASTM procedure C 702, Standard Practice for Reducing Field Samples of Aggregate to Testing Size. A mechanical splitter was used for this task. The size of the representative sample and respective test gradations were determined through the use of ASTM procedure C 136, Standard Method for Sieve Analysis of Fine and Coarse Aggregates. Each sample was separated into the following sieve gradations:

3/4" - 1/2"
1/2" - 3/8"
3/8" - No. 4
No. 4 - No. 8
No. 8 - No. 30
No. 30 - No. 200

Petrographic Analysis - A detailed petrographic examination is a necessary procedure to determine texture and composition. This analysis includes the determination of average grain size, grain shape, percentage of groundmass, void content and composition. Pieces of aggregate 3/8 inch in size are selected from which thin-sections are obtained based on the diversity of aggregate within a source. A modal analysis of individual grains within a thin-section is performed using a polarizing microscope, calibrated eyepiece and mechanical stage.

1. Grain size - Krynine (1948) defines texture as the interrelationship of individual particles of a rock and also establishes boundaries for rock texture. The average grain size in a fine grained rock is considered to be less than 0.0625 mm, and medium grained between 2.0 and 0.0625 mm. These parameters place the particles in a range between silt and sand. All of the samples analyzed thus far fall within these two categories. The size of the grains is thought to have a direct relationship on texture and the resulting friction values. Larger grains generally provide a greater degree of surface friction.

The size of individual grains is measured based on the point count method by moving a thin-section a specified distance using the mechanical stage on the polarizing microscope. After a grain is designated by the cross-hairs of the calibrated eyepiece, the stage is rotated so the grain can be measured at its widest point. A minimum of 200 grains are measured to ensure a confidence level of 95%. The standard deviation between grains is a measure of the heterogeneity in the sample. A higher range of grain size is favored with respect to polish resistance.

Material smaller than individual grains is commonly intercepted by the cross-hairs of the calibrated eyepiece. This material is termed groundmass which can be subdivided into two categories - cement and matrix. Cement is defined as fine or coarse material between grains; and matrix as the fine material commonly observed with floating grains. A small percentage of groundmass may prove beneficial in regard to polish susceptibility and friction values. The opposite may be true for large percentages of groundmass in the thin section.

2. Grain shape - Grain shape is also directly related to texture. Angular grains have a tendency to develop coarsely textured surfaces and provide a greater degree of resistance in comparison to rocks composed of anhedral grains. In a previous study performed by West, Johnson and Smith (1970), grain shape was determined through the use of Krumbein's pebble roundness chart. This chart has also proved useful in

this study. Based on a scale of 1 to 10, the majority of the grains are euhedral to subhedral in shape, measuring 1 to 4. Grain shape is measured concurrently with grain size.

3. Void content - Void content is based on the observation of voids viewed under the cross-hairs of the calibrated eyepiece while moving the mechanical stage. The percentage of voids is determined as the ratio of voids counted to the total point counts for the sample. It will be interesting to compare the effects of void content to the friction and polish numbers after additional data are obtained.

4. Composition - The quantity, type, and hardness of minerals has a direct effect on the polish susceptibility and resulting friction numbers of a given material. A large percentage of hard minerals in a soft matrix is expected to resist polishing, prevent uniform wear and provide a greater degree of frictional resistance.

The minerals of greatest abundance in dolomite rock are dolomite and calcite. Quartz and siliceous material are secondary. To distinguish the difference between calcite and dolomite, the thin-sections will be stained using a trypan blue solution. Dolomite stains blue whereas calcite remains colorless. The predominant minerals in limestone are calcite, dolomite and aragonite. Fiegl's solution and Alizarin Red S may be used in steps to dye aragonite - black and calcite - purple. Dolomite remains colorless.

Analysis of individual grains within the gravels and sandstones is expected to be more difficult because of the diversity of constituent rocks and minerals. In this case a polarizing microscope and petrographic techniques must be utilized to identify mineral composition.

Physical Properties - Existing data for the representative sources will be collected from the Indiana Department of Transportation. An attempt will be made to correlate the data to polish and friction values. The sodium sulfate test is commonly used to determine aggregate soundness. The results of this test are not easily duplicated and the freeze-thaw test is expected to eventually replace it. A correlation may be observed between aggregate soundness and the final polish/friction values. Two other factors related to aggregate quality, which will be considered, are specific gravity and absorption.

Acid-Insoluble Residue - The single most important test for carbonate aggregates may be the gradation, quantity and type of acid-insoluble residue. Hard minerals, such as quartz, have a tendency to renew polished surfaces as the edges of the rock break away. A ratio, not yet determined, of soft and hard minerals may actually reduce the polishing action of vehicular traffic. Carbonate aggregate containing high percentages of clay-like fines have been found to deteriorate rapidly with exposure to normal weathering in the presence of moisture; however, some clay material may be innocuous. Although not currently within the scope of this study, identification of clay type through x-ray diffraction techniques may be necessary.

The acid-insoluble test is performed by dissolving approximately 500 grams of aggregate in 6N HCl. Initially, it was assumed that the larger grain sizes would be the predominant insoluble material by weight and the material finer than the #200 sieve could be ignored. However, upon review of the data and evaluation of the intended technique, it was determined that, in most cases, the clay size material was the predominant insoluble fraction and must be considered. To date, the insoluble analysis has been completed on eighteen of nineteen samples using ASTM procedure D 3042, Standard Test Method for Insoluble Residue in Carbonate Aggregates.

Magnesium Content - A dolomite rock for aggregate use is defined as a carbonate rock containing 10.3 to 13.2 % elemental magnesium. The Indiana Department of Transportation currently uses elemental magnesium content as the acceptance criteria for dolomite sources with the minimum being 10.3%. Fifteen of the sources have been analyzed using ASTM procedure C 602, Specification for Agricultural Liming Materials. This is primarily an EDTA titration method and requires 0.5 grams of sample, ground to pass the #60 sieve. Data on the quantity of magnesium will be correlated with the polish and friction values to evaluate significance as an acceptance criteria.

British Wheel - This is a laboratory device used to polish coupons - a composite of 3/8 inch pieces of molded aggregate. The British Wheel simulates the polishing action of vehicular traffic on coarse aggregate (Figure 1). Fourteen curved molds, two samples consisting of ten molds and four control specimens, are inserted along the circumference of a steel wheel (Figure 2). A small pneumatic tire is placed in contact with the molds on the steel wheel. As the tire and wheel turn, grit and water are evenly applied to initiate a polishing action. The aggregate is polished for a period of approximately ten hours - the period of time with which equilibrium is expected to occur with most rock types. Results indicate the polish susceptibility of a given source of aggregate. This process is accomplished using ASTM procedure D 3319, Accelerated Polishing of Aggregates Using the British Wheel.

British Pendulum - Friction values on the molded aggregate are measured before and after polishing using a device called the British Pendulum (Figure 3). The Pendulum consists of a swinging arm with a rubber tip and a scale measuring in BPNs. BPNs are friction numbers which are unique to the British Pendulum test.

A coupon is cleaned, wetted and placed below the swinging arm of the Pendulum. The arm, adjusted in compliance with ASTM procedure E 303 specifications, Standard Method for Using the British Pendulum Tester, is allowed to swing so that the rubber tip barely contacts the aggregate surface. The level of friction encountered is measured on the Pendulum scale. Three measurements are performed on a single coupon with the average calculated as the BPN. These results indicate a reduction in frictional resistance indicative of a given source.

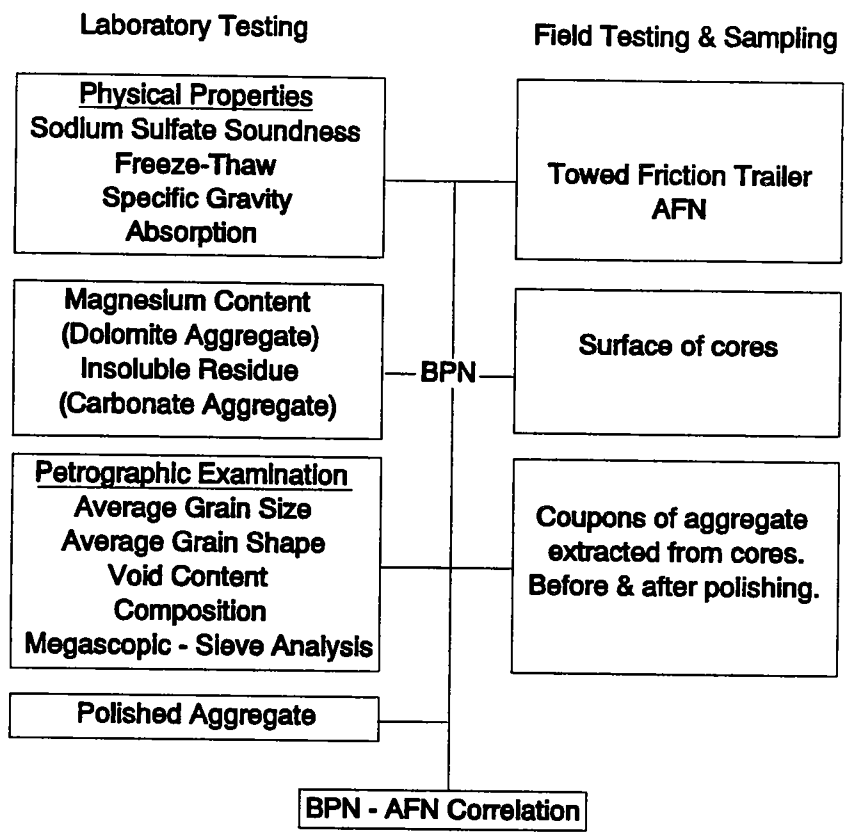

Figure 6. Data analysis

DRAFT SPECIFICATIONS AND REPORT PREPARATION

A report will be prepared outlining the final results of the project. With the assistance of the researchers, the Indiana Department of Transportation expects to draft specifications and implement a program which would allow expeditious evaluation of aggregate sources for use in bituminous surface overlays. Specifications will be based on the results of laboratory tests and a correlation between BPNs and AFNs.

SUMMARY

The primary objective of this highway research project is to develop procedures which will allow quick evaluation of new aggregate sources for use in bituminous surface overlays. Two methods, laboratory analysis and field testing, are being used to evaluate and correlate friction values of crushed aggregate sources and bituminous overlays.

Laboratory Testing entails analysis of aggregate sources through sieve analysis; a detailed petrographic analysis including average grain size, grain shape, percentage of groundmass, void content and composition; and a review of historical data concerning physical properties including sodium sulfate soundness, freeze-thaw, specific gravity and absorption. Other tests performed, which are sensitive to aggregate type, are acid-insoluble residue and magnesium content.

The largest pieces of crushed rock within a source from the Indiana #11 gradation, 3/8 inch, are molded into coupons and measured for friction level in BPN units using the British Pendulum. Coupons are then polished to equilibrium using the British Wheel and again tested for frictional resistance. The difference between the initial and final values gives an indication of aggregate polish susceptibility. The final value shows the polish threshold. A strong correlation between laboratory tests and BPN is expected.

Field Friction Testing and Sampling entails testing and analysis of friction properties for bituminous overlays and the embodied aggregate. The towed friction trailer is used to measure the AFN of a given section of overlay pavement and then the overlay is cored. After the surface of the core has been measured for friction level in the laboratory, the core is stripped of bitumen. The retrieved aggregate is tested for friction properties using aggregate coupons in the same manner as that for the raw crushed aggregate. The purpose is to determine a correlation between AFNs and BPNs of a given source. A report will be prepared and a program implemented to allow expeditious evaluation of aggregate sources for bituminous overlays.

REFERENCES

American Society for Testing and Materials, 1993, Standard Practice for Sampling Aggregates, Vol. 04.08, D 75-87: American Society for Testing and Materials, Philadelphia, PA, pp. 68-71.

American Society for Testing and Materials, 1993, Standard Practice for Reducing Field Samples of Aggregate to Testing Size, Vol. 04.02, C 702-87: American Society for Testing and Materials, Philadelphia, PA, pp. 368-371.

American Society for Testing and Materials, 1993, Standard Method for Sieve Analysis of Fine and Coarse Aggregates, Vol. 04.03, C 136-92: American Society for Testing and Materials, Philadelphia, PA, pp. 34-37.

American Society for Testing and Materials, 1993, Standard Test Method for Insoluble Residue in Carbonate Aggregates, Vol. 04.02, D 3042-92: American Society for Testing and Materials, Philadelphia, PA, pp. 715-718.

American Society for Testing and Materials, 1993, Standard Specification for Agricultural Liming Materials, Vol. 04.01, C 602-90: American Society for Testing and Materials, Philadelphia, PA, pp. 308-313.

American Society for Testing and Materials, 1993, Standard Test Method for Accelerated Polishing of Aggregates Using the British Wheel, Vol. 04.03, D 3319-90: American Society for Testing and Materials, Philadelphia, PA, pp. 376-379.

American Society for Testing and Materials, 1993, Standard Method for Using the British Pendulum Tester, Vol. 04.03, E 303-83: American Society for Testing and Materials, Philadelphia, PA, pp. 688-692.

Krynine, P.D.; 1948, The Megascopic Study and Field Classification of Sedimentary Rocks, in Sedimentary Rocks: Benchmark Papers in Geology, Vol. 15, Stroudsburg, PA, Dowden, Hutchinson & Ross, Inc., 1975, pp. 64-100.

West, T.R.; Johnson, R.B.; and Smith, N.M., 1970, Tests for Evaluating Gradation of Base Course Aggregates, National Cooperative Highway Research Program SBN 309-01885-4, Report 93: Highway Research Board, Washington D.C., p. 22.

Highway Construction Projects Have Legal Mandates Requiring Protection of Paleontologic Resources (Fossils)

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ABSTRACT

In common with other environmental disciplines such as archeology and biology, federal and state statutes require that highway construction project-related impacts on paleontologic resources (fossils) be mitigated to an insignificant level. Paleontologic resources are protected under the Federal Antiquities Act of 1906 (FAA) which forbids disturbance of any object of antiquity on federally owned lands and establishes criminal sanctions for destruction of objects of antiquities. In passing the Antiquities Act, Congress acknowledged that all objects of antiquity are valuable national resources. Like archeological remains, fossils are non-renewable natural resources. Once they are destroyed, they are gone forever and along with them is gone the only record of past life and past environments.

The Federal Highways Act of 1958 specifically extended the Antiquities Act to apply to paleontologic resources and authorized the use of funds appropriated under the Federal-Aid Highways Act of 1956 to be used for paleontologic salvage in compliance with FAA and any applicable state laws. The language in the Highways Act makes it clear that Congress intended that, to be in compliance with the Antiquities Act, highway construction projects must protect paleontologic resources.

The National Environmental Policy Act of 1969 (NEPA) requires that the environmental consequences of proposed highway construction projects be assessed. The courts have interpreted NEPA to also apply to state and local highway projects funded wholly or partly with federal monies. To comply with NEPA and FAA, the lead agency for a highway construction project has the responsibility of ensuring that contractors "shall not knowingly disturb, alter, injure, or destroy any scientifically important paleontological remains...."

To demonstrate compliance with these legal mandates to protect paleontologic resources during highway construction, we recommend that the guidelines published by the Society of Vertebrate Paleontology (SVP) for the mitigation of adverse impacts on paleontologic resources (SVP 1991) be adopted for each project. The SVP guidelines recommend that every project have a literature and archival review, field survey, and, if vertebrate fossils are identified within the construction corridor, a project-specific mitigation plan that includes monitoring of selected segments of the ROW to salvage any fossils encountered, and placement of identified and curated specimens into a permanent museum collection. The purpose of the paleontologic resource impact mitigation program should be to assist the federal or state highway agency, contractors, and subcontractors in complying with legislation protecting paleontologic resources that might be encountered during highway construction.

INTRODUCTION

Federal laws require protection of significant paleontologic resources, including fossil sites and fossil remains. In addition to federal legislation, many states have their own legislation protecting paleontologic resources. Marshall (1976), West (1991), Spencer (1993), and Martin (1993) have each reviewed federal and state laws protecting fossils. In this paper we provide a review of those laws specifically in reference to highway construction projects and outline appropriate mitigation measures which can be used to comply with these legal requirements.

Fossils are valuable, non-renewable natural resources which when destroyed by ground-disturbing construction projects are lost forever. Our only record of prehistoric life and much of our basis for interpreting paleoenvironments are lost with them. Paleontologic resources--like archeological, cultural, and historical resources--therefore deserve to be protected and preserved if we want the prehistoric record available for future generations.

Federal and state laws protecting fossils on public lands (and adjacent private lands) require that ground-disturbing construction projects mitigate negative impacts on paleontologic resources. Compliance with these laws protects scientifically and educationally valuable fossil specimens which can be salvaged and preserved for future generations to study and enjoy.

LEGAL MANDATES PROTECTING PALEONTOLOGIC RESOURCES

Most highway construction projects traverse a variety of rock types ranging from unconsolidated Quaternary sediments to Precambrian rocks and some of these are likely to be fossiliferous. Because of the potential for encountering fossil remains during excavation of some segments of a highway, there is a corresponding potential for project-related adverse impacts on legally protected and scientifically important paleontologic resources. Federal and state environmental statutes require that these project-related impacts on paleontologic resources must be mitigated to an insignificant level. The federal and some specific state statutes are reviewed briefly below. Although this section discusses the legal authority that mandates that fossils be protected, it is not intended to be either a comprehensive, precise, or technical analysis of these laws.

Federal Requirements. Paleontologic resources are afforded protection on federal land under federal environmental legislation, including, but not limited to, the following acts.

1) The Federal Antiquities Act of 1906 (FAA) (P.L. 59-209; 34 Stat. 225, 16 U.S.C. 432, 433) forbids disturbance or destruction of any object of antiquity on federal land without a permit issued by the responsible land-management agency. The specific language is that no person shall "appropriate, excavate, injure, or destroy...any object of antiquity, situated on lands owned or controlled by the Government of the United States...." FAA defines "objects of antiquity" broadly to include "objects of historic or scientific interest...." This Act also establishes criminal sanctions for unauthorized appropriation or destruction of antiquities. In passing the FAA, Congress acknowledged that all objects of antiquity are valuable national resources and deserve to be protected by law.

2) The Historic Sites Act of 1935 (P.L. 74-292; 49 Stat. 666, 16 U.S.C. 461 <u>et seq.</u>) declares it a national policy to preserve objects of national significance for public use and gives the Secretary of the Interior broad powers to execute this policy, including criminal sanctions.

3) The Federal Highways Act of 1958 (P.L. 85-767; 72 Stat. 885, 23 U.S.C. 101 et seq.) specifically extended the Antiquities Act to apply to paleontologic resources and authorized the use of funds appropriated under the Federal-Aid Highways Act of 1956 (P.L. 86-657; 74 Stat. 522) to be used for paleontologic salvage in compliance with FAA and any applicable state laws. The language in the Highways Act makes it clear that Congress intended that, to be in compliance with the Antiquities Act, highway construction projects must protect paleontologic resources.

4) The National Environmental Policy Act of 1969 (NEPA) (P.L. 91-190; 31 Stat. 852, 42 U.S.C. 4321-4327) requires that important natural aspects of our national heritage be considered in assessing the environmental consequences of any proposed undertaking.

5) The Federal Land Policy Management Act of 1976 (FLPMA) (P.L. 94-579; 90 Stat. 2743, U.S.C. 1701-1782) requires that public lands be managed in a manner that will protect the quality of scientific and archeologic values.

State Requirements. In addition to the federal legislation discussed above, many individual states have their own legislation protecting fossils. As examples, we summarize below laws in the states of Idaho, Washington, Oregon, and California. For discussions of other state laws see West (1991).

In the state of Idaho vertebrate fossil sites are protected on all state lands under Idaho State Regulations 67-4119 through 67-4122. Permits for excavation of fossil remains must be obtained from the Idaho State Historical Society and the society's permission must be obtained before vertebrate fossils can be removed from the state.

In Washington State the Department of Natural Resources regulates collecting of all geological and paleontological materials on state lands under the authority of Section 155, Chapter 255, Laws of 1927 and Revised Codes of Washington (RCW) 79.01.616. An amendment to antiquities legislation in 1989 (Substitute Senate Bill No. 5807) extended the enforcement to include private lands. Specimens collected on public lands must remain in the state; those collected on private lands may be removed from the state (West 1991).

Paleontologic resources in Oregon (including fossil plants and wood, fossil invertebrates, and fossil vertebrates) are protected under Oregon Revised Statute (ORS) 273.705, Oregon Administrative Rules (OAR) 660-01 through 660-40. These regulations require that local governments form environmental management "goals", which must be approved by the Land Conservation and Development Commission (LCDC). "All of the text under the heading *Goal* is mandatory and has the force of law" (LCDC 1990, p. 1). Goal 5, entitled *Open Spaces, Scenic and Historic Areas, and Natural Resources*, requires protection of natural resources, including "paleontological features" (LCDC 1990, p. 6; OAR 660-15-000, 660-16-000 through 660-16-025, covered under ORS 183).

In California, paleontologic resources are protected on state land under state environmental legislation, including, but not limited to, the following acts:

1) California Environmental Quality Act of 1970 (CEQA) (13 Public Resources Code: 21000 et seq.) requires public agencies and private interests to identify the environmental consequences of their proposed projects on any object or site significant to the scientific annals of California (Division I, Public Resources Code: 5020.1 [b]).

2) Guidelines for the Implementation of CEQA, as amended May 10, 1980 (14 California Administrative Code: 15000 et seq.), define procedures, types of activities, persons, and public agencies required to comply with CEQA and include definitions of significant effects on a site containing fossils (Section 15023, Appendix G [j]).

3) Public Resources Code, Section 5097.5 (Stats. 1965, c. 1136, p. 2792), defines any unauthorized disturbance or removal of fossil remains or sites located on public land as a misdemeanor.

To comply with the legal mandates summarized above (particularly FAA, NEPA, and FLPMA), the federal lead agency for a highway construction project, as well as any cooperating federal agencies such as the U.S. Bureau of Land Management (BLM) and/or U.S Forest Service (USFS), and many state, county, and city agencies, have the responsibility for ensuring the protection of paleontologic resources in areas under their respective jurisdictions. In some instances in the past where a federal agency was the lead agency for a project that included federal land, federal guidelines have also been applied to both state and private lands. For example, as the result of recent litigation, it is now BLM policy in some areas to enforce NEPA guidelines along the entire length of a ROW if a BLM parcel is essential to the project, even if the parcel is only a very minor component of the ROW (G. Portillo, BLM, Riverside, California; personal communication to L.A. Spencer, 1991).

ASSESSING POTENTIAL IMPACTS

During highway construction, both adverse and beneficial impacts on significant paleontologic resources can result. Adverse impacts on paleontologic resources occur because the earth surrounding and protecting the fossils is disturbed, allowing the fossils to be destroyed. This can occur directly by earth-moving equipment, or indirectly by allowing weathering agents to reach previously buried specimens. Mechanical destruction can occur not only during actual excavation of the roadway, but also during construction of access roads, clearing and grading of the ROW, and constructing stockpile areas and borrow pits. Indirect adverse impacts occur where weathering agents are allowed to reach specimens previously naturally protected by burial. This can occur by fracturing the ground adjacent to construction and percolation of ground water through the enhanced permeability created. Additionally, changes in surface grade modify the drainage pattern and allow erosion of previously protected areas; increased erosion exposes previously protected fossils to surface weathering and rapid destruction.

Implementation of a well designed mitigation plan prepared by professional and experienced paleontologists can result in a substantial reduction in the severity of adverse construction-related impacts. A well designed mitigation program can also provide some beneficial impacts by preserving previously unexposed fossils and associated data in a public museum where they are available for future study.

GUIDELINES FOR MITIGATING ADVERSE IMPACTS

Until recently, there have been no formal guidelines regarding the measures to be employed in mitigating potential adverse impacts on paleontologic resources. Often, the measures recommended by any paleontologist were routinely accepted by regulatory agencies. However, there have been disagreements among individual paleontologists regarding the measures required to mitigate adverse impacts on paleontologic resources (see Spencer and Lander 1991). As a result, the Society of Vertebrate Paleontology (SVP) has recently released "standard" guidelines for mitigating adverse impacts on vertebrate fossils. The SVP "*Standard Measures for Assessment and Mitigation of Adverse Impacts to Nonrenewable Paleontologic Resources*" (SVP 1991) represents the consensus of vertebrate paleontologists on appropriate measures for mitigating adverse impacts on vertebrate fossils. To minimize individual interpretations on appropriate levels of mitigation, we recommend that the SVP standard measures be used both to evaluate the potential adverse impacts to paleontologic resources and to mitigate and protect those resources during construction. Because the SVP measures are the only published standard for assessing adverse impacts on paleontologic resources and for formulating specific mitigation plans, a review of each SVP guideline and its rationale is presented below.

Significance and Sensitivity. In common with other environmental disciplines such as archeology and biology, where every cultural artifact and every specimen of a threatened or endangered species is held to be significant unless shown to be otherwise, the Society of Vertebrate Paleontology considers every fossil specimen significant, unless demonstrated otherwise, and, therefore, protected by environmental statutes. This position is held because vertebrate fossils are usually uncommon and only rarely will a fossil locality yield a statistically significant number of specimens representing the same species. In most cases, each vertebrate fossil specimen found will provide additional important information about the characteristics, ecology, or distribution of that fossil species.

A rock unit (such as a formation, member, or bed) known to contain significant vertebrate fossils is considered "sensitive" to adverse impacts if there is a high probability that ground-disturbing or earth-moving activities will destroy fossil vertebrate remains in that rock unit. This definition of sensitivity differs fundamentally from that for cultural resources:

> "It is very important to make the distinction between archaeologic resource sites and paleontologic resource sites when defining sensitivity. Archaeologic site boundaries define the limit of the extent of the resource. Paleontologic sites, however, serve [only] as indicators that the sedimentary unit or formation in which they are found is fossiliferous. The boundaries of an entire fossiliferous formation, therefore, define the limits of paleontologic sensitivity in a given region" (SVP 1991).

The distinction between archeologic and paleontologic (fossil) sites is important. Most archeologic sites have a surface expression that allows their precise geographic location to be determined. Fossils, on the other hand, form as an integral component of the rock unit below the ground surface, and will often not be observable unless exposed by erosion or human activity. A paleontologist usually will not know the quality or quantity of fossils present prior to exposure of the rock unit as a result of natural erosion processes or earth-moving activities.

Prior to exposure of portions of the rock unit, a paleontologist may not be able to determine the location of fossils within the rock unit except at the surface of an exposure. Fossils are seldom uniformly distributed within a rock unit. Much of a rock unit may lack fossils, but concentrations of fossils may exist. Even within a fossiliferous portion of the rock unit, fossils may occur in local concentrations. For example, Shipman (1977, 1981) excavated a fossiliferous site using a three dimensional grid and removed blocks of rock matrix of a consistent size. Although this site was already known to be richly fossiliferous, only 17% of the blocks contained fossils due to the local concentrations in the distribution of fossils. This study demonstrates the physical basis for the difficulty in predicting the location of fossils prior to excavation.

Since it is not usually possible to determine in advance where fossils are located without actually disturbing a rock unit, monitoring of excavation by a paleontologist during construction increases the probability that fossils will not be destroyed, but rather that they will be discovered and preserved. Preconstruction mitigation measures such as surface prospecting and collecting will not prevent adverse impacts on fossils due to the frequent absence of fossils at the surface.

The nonuniform distribution of fossils is essentially universal and many paleontologic resource assessment and mitigation reports conducted in support of environmental impact statements (EIS), environmental impact reports (EIR), and mitigation plan summary reports document similar findings (see Lander 1989; Reynolds 1990; Spencer 1990; Spencer and Fisk 1991; Fisk, Spencer, and Whistler 1994a, 1994b, 1994c; and references cited therein). In fact, most fossil sites recorded in reports of impact mitigation (where construction monitoring was implemented) had no previous surface expression. Because the presence or location of fossils within a rock unit cannot be known without exposure resulting from previous erosion or excavation, the entire rock unit is assigned the same level of sensitivity based on previously recorded fossil occurrences.

The SVP considers a rock unit to be sensitive to adverse impacts if it is known to contain vertebrate fossils and considers those adverse impacts to be mitigated to an insignificant level if the fossils and contextual geologic data are salvaged and placed into a public museum collection. To mitigate these impacts, it is necessary to develop a paleontologic resource management program that: 1) identifies the sensitive rock units and fossil sites to be disturbed by construction; 2) locates and preserves the fossils encountered before and during ground-disturbing construction activities; and 3) incorporates the fossils into a public museum collection. These three tasks correspond to pre-construction, construction, and post-construction activities.

Preconstruction Assessment. Under SVP guidelines, prior to construction, an assessment must be conducted of the paleontologic sensitivities of the rock units traversed by a proposed construction project. The assessment must include a review of the published and unpublished geologic and paleontologic literature covering the project area, as well as a review of museum collections and site maps and records for data on the fossil-bearing rock units to be crossed by the proposed construction project, and a field survey to verify the present condition of previously recorded fossil sites and inspection of exposures along the ROW for additional, new sites.

The purpose of the literature and archival reviews is to identify segments of the highway ROW and portions of the temporary work areas which are underlain by rock units with a high probability of containing significant paleontologic resources and to develop an inventory of these resources. The literature review should be as thorough as possible and should include a computer search for both geologic and paleontologic articles relating to each stratigraphic unit which will be encountered during construction. Archival reviews should be conducted at all museums in the vicinity of the project and all others suspected of having collections of fossils from the general area. This search should include publicly and privately owned museums, universities and colleges, and government agencies (such as the U.S. Geological Survey and state geological surveys). On the West Coast of the United States, a thorough archival search should include as a minimum the following institutions, all of which contain large collections of fossils: in California, the University of California Museum of Paleontology at Berkeley, the Natural History Museum of Los Angeles County in Los Angeles, University of California at Riverside, and University of California at Davis; in Oregon, University of Oregon Condon Museum of Paleontology in Eugene and the Pacific Northwest Museum of Natural History in Ashland; in Washington, University of Washington Burke Museum (UW) in Seattle; and in Idaho, Idaho State University Museum of Natural History in Pocatello.

Because of the high cost relative to the potential information to be gained, out of state museums some distance from the project may not need to be checked. Past experience has shown us that most collections at midwest and eastern institutions are either duplicated by collections at western institutions or were collected many years ago and lack sufficient stratigraphic and geographic data to be helpful. Even in those instances when an eastern institution has adequate site data, it is likely that western institutions also have a representative collection from the same rock units or even the same localities. Therefore, the information contained in distant institutions probably would not significantly change the mitigation measures recommended for most projects.

If during the archival literature search no references are found to significant paleontologic resources on or near the ROW and none of the institutions surveyed have significant fossils from the area, then it must be concluded that the rock units crossed by the highway have a low probability of containing significant paleontologic resources. Accordingly, no field survey need be conducted. In addition, monitoring during construction should not be recommended. Rather, construction crews should be instructed that if fossils are encountered, construction should be temporarily halted or moved forward of the fossil site and the project paleontologist immediately notified.

On the other hand, if fossil localities have been previously reported in the vicinity of or actually on the ROW, a field survey should be conducted to verify that sensitive rock units identified during the literature and archival reviews cross the ROW at the points previously mapped, to document the present condition of any previously recorded fossil site, to look for any previously unrecorded fossils sites in the ROW, and to identify areas where special mitigation measures need to be implemented prior to construction to avoid construction delays. If the field survey verifies that a fossiliferous rock unit is present on or near the ROW, it can be confidently judged that there is a high probability that highway construction activities will cause adverse impacts to significant paleontologic resources. Therefore, the paleontologic resource impact mitigation plan should be prepared to reduce these potentially adverse impacts to an insignificant level.

The preconstruction sensitivity assessment should be conducted well in advance of actual construction startup in order to use this information in construction scheduling and planning. By adopting and implementing a mitigation plan prepared well in advance of construction, potential adverse impacts on paleontologic resources resulting from construction of the project can be minimized and possible construction delays can be avoided.

Preparing a Mitigation Plan. Since paleontologic resources are protected by federal and state laws, if the preconstruction sensitivity assessment determines that it is probable that there will be adverse impacts on significant paleontologic resources any where along the highway ROW, a project-specific mitigation plan must be developed to reduce the potential adverse impacts to an insignificant level. The mitigation plan should involve construction worker education, monitoring of selected intervals of the ROW during excavation to salvage any fossils encountered, and sampling fossiliferous spoils for small vertebrate fossils. More specifically the mitigation program should include the following:

1) Prior to construction, all personnel should be given environmental training, stating that fossil resources may be encountered during construction. Further, workers should be instructed that, if fossils are seen in areas without a paleontologic monitor, the project paleontologist should immediately be notified, and construction should be temporarily halted or moved forward of the fossil-bearing section until a determination is made of their significance and a plan of action is formulated.

2) A field paleontologist should monitor all ground-disturbing activities in those sections of the ROW identified during the literature/archival reviews and field survey as having a high potential for containing paleontologic resources. In areas of highest paleontologic sensitivity, the paleontologist should monitor not only excavation, but also examine freshly exposed surfaces during clearing and grading operations. The paleontologist should salvage fossils exposed during construction and record stratigraphic, lithologic, and geographic data. As necessary, fossiliferous sediment samples should be taken from the spoils to determine if small vertebrate fossils are present. The paleontologic monitor should assign a field number to each fossil site discovered and record in his/her field notebook the geographic location and associated geologic information. If the paleontologic monitor notes either an unusually large number of fossils or a highly significant specimen being exposed by earth-moving operations, he/she should have permission to request that construction be stopped or temporarily delayed to allow the project paleontologist, environmental inspector, and chief construction inspector, in consultation with the managing agency representative, to make a determination of site-specific mitigation requirements. Depending on the specific circumstances, the mitigation procedure could involve either moving construction forward, away from the fossil locality , to allow paleontologists to carefully excavate the fossil site, or excavating through the fossil site, destroying some fossils, and then salvaging fossils from adjoining portions of the site.

3) Excavation spoils found to contain small vertebrate fossils during monitoring should be processed according to the standard procedures described by SVP (see also Spencer and Fisk 1991). The bagged and labelled fossiliferous concentrate produced by screening is transported to the laboratory for final processing. Here the concentrate is examined under magnification and small fossil specimens, such as small bones and teeth, are picked out of the concentrate.

4) Each salvaged fossil should be preliminarily identified to the lowest taxon possible and curated into a publicly available museum collection.

Monitoring during Construction. Because the subsurface location of fossils cannot be determined prior to excavation, the SVP (1991) recommends that the mitigation plan require monitoring of excavation by a qualified paleontologist:

> "In sedimentary units with a known potential [to contain fossils], a qualified paleontologic monitor is [to be] present during ground-disturbing activities 100% of the time unless the qualified project paleontologist determines that reduced monitoring is adequate until fossils are found."

The monitor must be present during excavation to salvage fossils uncovered by earth-moving activities. If a known fossil site will be impacted by earth-moving activities, the surface collection of fossil specimens from this site may begin prior to construction to minimize delay and to partially mitigate the adverse impact on the site. However, specimens exposed at the surface generally represent only a very small percentage of the specimens present at the site. Subsequent earth-moving activities may impact the larger, unexposed portion of the resource and require additional mitigation.

To adequately protect significant fossils, a qualified paleontologic monitor must be present during all ground-disturbing construction activities through sensitive rock units to salvage the fossils uncovered by these activities. Construction personnel and environmental inspectors cannot perform this task because they are obligated to their own tasks. In addition, these personnel also lack the necessary experience required to recognize fossils and salvage them. Only large specimens are conspicuous enough to be recognized by construction personnel or environmental inspectors. While large fossils, such as mammoth tusks, dinosaur bones, or petrified logs, are impressive, they often are not as important scientifically as smaller fossils, such as small rodents, which usually convey more information regarding geologic time and paleoenvironments. In addition, in many cases, fossils are similar in color to the surrounding rock matrix and are unlikely to be observed by untrained personnel. There is also the potential for fossil remains, when observed by unsupervised construction personnel, to be collected and placed in private, personal collections, or even sold.

Preserving Associated Geologic Data. As noted above, the SVP (1991) considers adverse impacts on vertebrate fossils to be mitigated if the specimens and associated geologic data are preserved. Associated or contextual geologic data may include measured stratigraphic position, depositional environment, associated plant and invertebrate fossils (paleocommunity), and magnetostratigraphic and other geochronologic information. All of these data may not be present at every site, but as much associated data as possible should be salvaged as part of the mitigation plan. Notes on stratigraphic position, depositional environment, and other associated geologic data are considered preserved when they are recorded in the permanent field notebooks of the paleontologic monitor. In addition, associated plant and invertebrate fossils should be collected and preserved as part of the public museum collection. In instances where it is particularly significant, magnetostratigraphic and geochronologic information should also be collected. To aid in interpreting the paleoenvironment and depositional setting of vertebrate fossils salvaged during excavation, sediment samples can also be collected for palynology. At each site yielding megafossils, a clean rock exposure should be sampled to reduce the possibility of contamination with modern pollen. Samples should be taken at the same stratigraphic horizon and in close proximity to the megafossils. Judgments regarding the collection of additional associated geologic data must be made by the paleontologic monitor based on site-specific criteria.

Recovering Small Vertebrate Fossils. The scientific value of any identifiable remains of small vertebrate fossils recovered by processing of fossiliferous sediments is equivalent to or greater than the scientific value of the larger vertebrate fossils recovered during monitoring because small mammals often exhibit greater change through a stratigraphic sequence and, therefore, may be more age diagnostic. Moreover, small mammals are more sensitive to environmental parameters, and thus their fossil remains can be more useful in reconstructing paleoenvironments.

Because of their size, small vertebrate fossils are often not apparent in the field and must be recovered by screening of the sediment matrix. The SVP (1991) has established two cubic meters as the standard sample size for sediment processing to recover small vertebrate fossils. This sample size is based in part on previous research regarding the amount of sediment that must be processed before the number of newly recorded species in the sample begins to diminish rapidly with additional processing (Curtaining Effect or Law of Diminishing Returns). Preston (1948, 1962a, 1962b) has demonstrated that modern species abundance has a log-normal distribution. For example, 100 specimens of one common or abundant species must be collected before even one individual of a rare species will be collected, or 1,000 to 10,000 specimens must be collected before even one specimen of a very rare species will be collected in a modern biota. The same is true for fossil species. Wolff (1975) concluded that the following minimum sample characteristics are needed for analysis of a fossil mammalian fauna: "...to accumulate the total number of mammalian taxa once living in an area, at least 5,000-10,000 kg of bulk sediment samples may be needed...." Unfortunately, such large samples are costly and time consuming to obtain and process. After review of these data, the SVP (1991) concluded that a 3000 kg sample (roughly two cubic meters or 6,000 pounds) will yield satisfactory, if not comprehensive, results by salvaging representatives of <u>most</u> of the taxa buried at the site being sampled.

Curation and Storage. Adverse impacts on paleontologic resources are not considered to be completely mitigated until all of the specimens salvaged during construction are curated and accessioned into a permanent and publicly available museum collection. Specimens salvaged but not properly curated into a museum collection remain unavailable to the scientific community. Therefore, the SVP (1991) recommended that all specimens be catalogued, preliminarily identified, and placed into a retrievable storage, and that a complete set of field notes, geologic maps, and stratigraphic sections accompany the collections. A final report, which includes methods, stratigraphic and lithologic descriptions, taxonomic identifications, faunal and floral lists, paleoenvironmental and age interpretations, further analysis and significance of the specimens collected, and an inventory of the recovered remains, must also accompany the collections. Faunal and floral lists should be compiled by rock unit and site, and community associations should be analyzed where possible.

Compliance. Implementation of SVP standard mitigation measures, along with submission of periodic progress reports and a final report to the appropriate regulatory agencies, will demonstrate compliance, both during and following construction. When a copy of the mitigation program final report is filed with the lead agency responsible to oversee the construction project, its receipt and acceptance indicates completion of the impact mitigation program and project compliance with environmental statutes, guidelines, and authorizing agency mitigation measures developed during the assessment phase of the project.

SUMMARY

Federal and state statutes require that highway construction project-related impacts on paleontologic resources (fossils) be mitigated to an insignificant level. Compliance with these laws can be accomplished in part by identifying segments of the highway right-of-way (ROW) underlain by formations with a high potential for containing fossil remains that could be disturbed by construction. Based on this identification, mitigation measures can be formulated to substantially mitigate the disturbance by recovering scientifically important fossil remains and contextual geologic data that otherwise would be lost due to construction, and preserving them in a public museum.

With the implementation of a well-designed mitigation plan, adverse impacts on significant paleontologic resources can be minimized and, instead, beneficial impacts can result from preserving fossils and associated geologic data for future study. The mitigation plan should call for construction worker education, monitoring of selected intervals of the ROW during excavation to salvage any fossil remains encountered during all phases of construction involving ground-disturbing and earth-moving activities, and sampling fossiliferous spoils for small vertebrate fossils. Adverse impacts are not considered completely mitigated until all fossil specimens salvaged are preliminarily identified, curated, and accessioned into a permanent and publicly available museum collection. Complete sets of field notes, geologic maps, stratigraphic sections, and a final report, which includes methods, results, faunal and floral lists, analysis, significance, and an inventory of the recovered remains, must also accompany the collections. When a copy of the mitigation program final report is filed with the lead agency responsible for the highway construction project, its receipt and acceptance will indicate compliance with applicable federal and state environmental statutes.

LITERATURE CITED

Fisk, L. H., L. A. Spencer, and D. P. Whistler, 1994a, Paleontologic resource impact mitigation on the PGT-PG&E Pipeline Expansion Project: Final Report, Volume I: Idaho, Washington, and Oregon: Unpublished report prepared for Bechtel Corporation, San Francisco, CA, by Paleo Environmental Associates, Altadena, CA, 123 p.

Fisk, L. H., L. A. Spencer, and D. P. Whistler, 1994b, Paleontologic resource impact mitigation on the PGT-PG&E Pipeline Expansion Project: Final Report, Volume II: California: Unpublished report prepared for Bechtel Corporation, San Francisco, CA, by Paleo Environmental Associates, Altadena, CA, 72 p.

Fisk, L. H., L. A. Spencer, and D. P. Whistler, 1994c, Paleontologic resource impact mitigation on the PGT-PG&E Pipeline Expansion Project: Final Report, Volume III: Confidential Locality Maps: Unpublished report prepared for Bechtel Corporation, San Francisco, CA, by Paleo Environmental Associates, Altadena, CA, 47 p.

Fisk, L. H., L. A. Spencer, E. B. Lander, E. P. Gustafson, and H. M. Wagner, 1994, Beneficial impacts of large construction projects on paleontologic resources: results from construction of the PGT-PG&E Pipeline Expansion Project, WA-OR-CA: Proceedings of the Fourth Conference on Fossil Resources, "Partners in Paleontology: Protecting our Fossil Heritage", Colorado Springs, CO, 2 p.

Lander, E.B., 1989, Interim paleontologic resource technical report, Eastside Reservoir Project Study -- Phase 1, Riverside County, California: Unpublished report prepared for Metropolitan Water District of Southern California, Los Angeles, CA, by Paleo Environmental Associates, Altadena, CA, 50 p.

Marshall, L.G., 1976, Paleontological salvage and federal legislation: Journal of Paleontology, v. 50, p. 346-348.

Martin, J.E., 1993, Management of vertebrate paleontological resources [Draft]: Oregon State Office, Bureau of Land Management, Portland, OR, 138 p.

Preston, S.W., 1948, The commoness, and rarity, of species: Ecology, v. 29, no. 3, p. 254-283.

Preston, S.W., 1962a, The canonical distribution of commoness and rarity: Part I: Ecology, v. 43, no. 2, p. 185-215.

Preston, S.W., 1962b, The canonical distribution of commoness and rarity: Part II: Ecology, v. 43, no. 3, p. 410-432.

Reynolds, R.E., 1990, Paleontologic mitigation program Midway-Sunset Cogeneration Project, Kern County, California: Unpublished report prepared for Midway-Sunset Cogeneration Company, Taft, CA, 193 p.

Shipman, P., 1977, Paleoecology, taphonomic history and population dynamics of the vertebrate assemblage from the middle Miocene of Fort Turnan, Kenya: Unpublished Doctoral Dissertation, New York University, NY.

Shipman, P., 1981, Spatial distribution of fossils in sediments: pp. 65-98, in P. Shipman, Life history of a fossil, an introduction to taphonomy and paleoecology: Harvard University Press, Cambridge, Massachusetts, 222 p.

Spencer, L.A., 1990, Paleontologic resource assessment, Midway Sunset Cogeneration Project, Natural Gas Pipeline: Kern County, California: Unpublished report prepared for Midway-Sunset Cogeneration Company, Taft, CA, by Paleo Environmental Associates, Altadena, CA, 16 p.

Spencer, L.A., 1993, Paleontologic resources program policy manual [Draft]: Utah State Office, Bureau of Land Management, Salt Lake City, UT, 127 p.

Spencer, L.A., and E.B. Lander, 1991, Preliminary evaluation of paleontologic resource management programs: Mojave and Kern River natural gas pipelines: Unpublished report prepared for Chambers Group, Inc., Santa Ana, CA, by Paleo Environmental Associates, Altadena, CA, 29 p., plus appendices.

Spencer, L.A., and L. H. Fisk, 1991, Paleontologic resource assessment/mitigation plan, PGT-PG&E Pipeline Expansion Project, Volume 1, PGT Section, Idaho, Washington and Oregon: Unpublished report prepared for Pacific Gas Transmission Company, San Francisco, CA, by Paleo Environmental Associates, Altadena, CA, 61 p.

Society of Vertebrate Paleontology, 1991, Standard measures for assessment and mitigation of adverse impacts to nonrenewable paleontologic resources: Society of Vertebrate Paleontology News Bulletin, no. 152, p. 2-5.

West, R.M., 1991, State regulation of geological, paleontological, and archaeological collecting - 1991: Unpublished manuscript, American Museum of Natural History, New York, NY, 30 p.

Wolff, R.G., 1975, Sampling and sample size in ecological analyses of fossil mammals: Paleobiology, v. 1, p. 195-204.

BV-116: The Bridge by Addendum: Wetlands Dictate a Change in Design

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Abstract

The Beaver Valley Expressway was a major expansion project of the Pennsylvania Turnpike Commission. Designed and constructed on a fast-track schedule, the completed 16.5 mile roadway was opened to traffic in 1993. The central section of the roadway passes through a major, glacially-formed wetland. The original design called for crossing the wetland with embankment and installing a box culvert to ensure hydraulic connection between the bisected portions. While the contract for construction was out for bids, state environmental agencies determined that the wetland would be in jeopardy unless the box culvert was replaced with a bridge with an opening of one hundred feet at wetland elevation. In response, Addendum No.3 was prepared to provide a bridge at this location.

Geotechnical design for the new bridge structure was based on available subsurface information. Significant settlement had previously been estimated at this area, and wick drains had been provided in the construction contract by special provision. Stability was to be achieved by undercutting the embankment toe area and installing rock embankment. It was decided to construct a three-span continuous structure, with the piers founded on piles to bedrock and the abutments founded on spread footings in the embankment. A simple surveying program was developed to monitor settlement and provide some control over the contractor's activities.

At the end of the contract-allotted time for settlement, the contractor was allowed to construct the abutments even though essentially complete consolidation was not reached. When continued movement of the abutments required their modification prior to setting of the steel, it was obvious additional measures were needed. Additional borings were drilled, and piezometers and slope inclinometers were installed. New survey points were established. New estimates of settlement and stability were made, leading to modifications of both embankment and structure design.

Settlement at the abutments continues, having exceeded eight inches since the surveys were started and three inches since the deck was poured. However, the bridge and approaches have successfully handled the traffic since opening of the expressway.

Acknowledgments

The author appreciates the support provided by both his own company, Michael Baker Jr, Inc., and the Pennsylvania Turnpike Commission in the preparation and presentation of this paper. However, the views and conclusions contained herein are those of the author and should not be interpreted as necessarily representing the official policies or recommendations of either Michael Baker Jr, Inc. or the Pennsylvania Turnpike Commission.

The stage is set

The Beaver Valley Expressway (Toll 60) is a 16.5 mile long, four-lane divided highway running through Beaver and Lawrence Counties in western Pennsylvania. Bridge BV-116 is located immediately south of the county line which divided the project roughly in half. The bridge is a three-span, continuous, dual structure over an unnamed tributary to Jordan Run.

The project was undertaken by the Pennsylvania Turnpike Commission, with Michael Baker Jr, Inc. as its general consultant and Dick Corporation as the project manager. Final design and construction of the project was performed in six sections, numbered from 40 to 45 from south to north. Section 42, which contains BV-116, includes 2.63 miles of Toll 60, two interchange facilities with over 28,000 feet of ramps, and 9,700 feet of two-lane roadway relocation. Two other dual structures and four single structures are also a part of this section. The section was designed by Buchart-Horn, Inc. and its geotechnical subconsultant, GeoMechanics, Inc., and constructed by Trumbull Corporation.

Two baseline geologic references for the area identify the soils at the BV-116 site as nearly level, deep, and poorly drained within an irregular depression in a glaciated upland - a marsh (USDA/SCS, 1982 and DeWolf, 1929). The soil survey specifically identifies the soil as "suited to wildlife habitat." (USDA/SCS, 1982). The 7 1/2 minute quadrangle of the area and the soil survey both indicate that the area surrounding this local depression has been extensively strip mined for coal. Locally, the Upper Freeport Coal reached thicknesses exceeding 7 feet, was only gently dipping, and was readily accessible from hillside outcrops. This stripping, combined with earlier sand and gravel operations in the area, disrupted the original drainage of Jordan Run and promoted the establishment of a wetland that could be considered the best on the Beaver Valley Expressway, from an environmental perspective.

During preliminary design, an Environmental Assessment (EA) and Composite Technical Basis Report were prepared by Gannett Fleming Transportation Engineers, Inc., and completed in September 1989. As a part of those studies, this area was described as a palustrine mixed emergent and scrub/shrub, broad-leaved deciduous wetland in the Jordan Run headwater area. It covered an area of approximately 53 acres, with dominant species including cattails, rushes, sensitive fern, woolgrass, blue vervain, swamp dogwood, hawthorn, viburnum, black cherry, red maple, and shingle oak. The preferred alignment would cross this largest identified wetland in the corridor on fill. The crossing was proposed where the wetland was narrowest. In this manner, wetland impact would be minimized to the extent possible. See Figure 1 below.

In response to the EA, the PA Fish Commission raised concern over the large amount of stream involvement and loss associated with the project. It was assured that final design would incorporate all possible measures to reduce culvert length. The PA Game Commission raised concerns due to destruction of habitat and disruption of local populations. It was assured that wetland impacts and mitigation would be further studied during final design. The USEPA reminded the Commission that the final design needed to avoid and minimize adverse impacts to streams, wetlands, and any other water of the US to the maximum extent possible.

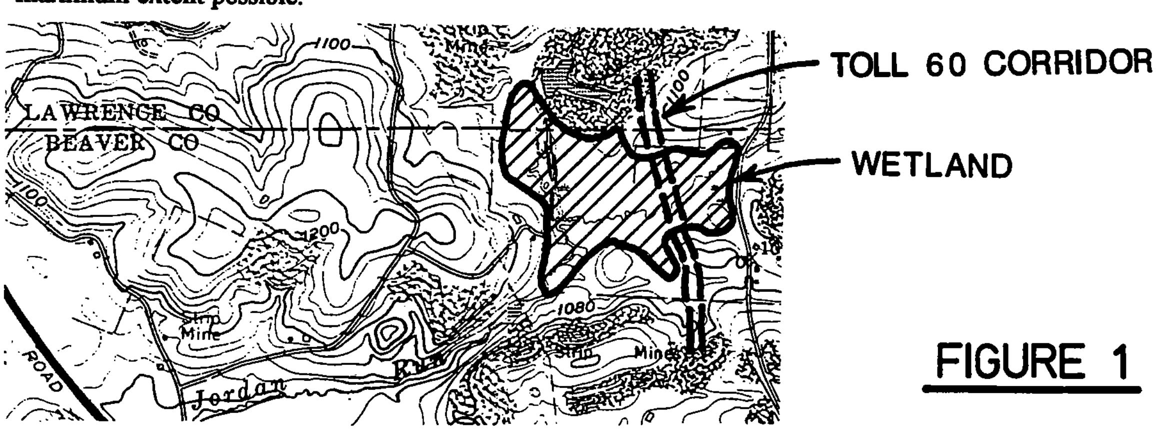


FIGURE 1

Geotechnical concerns are addressed

Final design of Section 42 was started in November of 1988, with plan and specification submittal anticipated to allow construction bidding during the spring of 1990.

The proposed roadway alignment would essentially bisect the wetland, with the grade averaging forty five feet above the existing ground surface. The design called for a full embankment section with 2:1 (H:V) side slopes and a culvert to hydraulically connect both halves of the wetland, with minor redefinition of the channel at both ends. The culvert was to be a 319 foot-long, six-by-ten foot box structure on a two-foot thick bed of coarse aggregate. The preliminary cost estimate for the culvert was .28 million dollars.

A series of five borings were drilled along the centerline of the culvert to better define the subsurface conditions. The soils were determined to be approximately 70 feet thick. Some stratification was apparent from the borings, allowing an idealized section to be constructed. The subsurface was characterized as having four primary layers:

6 feet - silty clay	N(avg.) = 3
27 feet - silty sand	N(avg.) = 15
27 feet - clay	Cc = 0.22, e = 0.4936
10 feet - sand and gravel	N(avg.) = 30

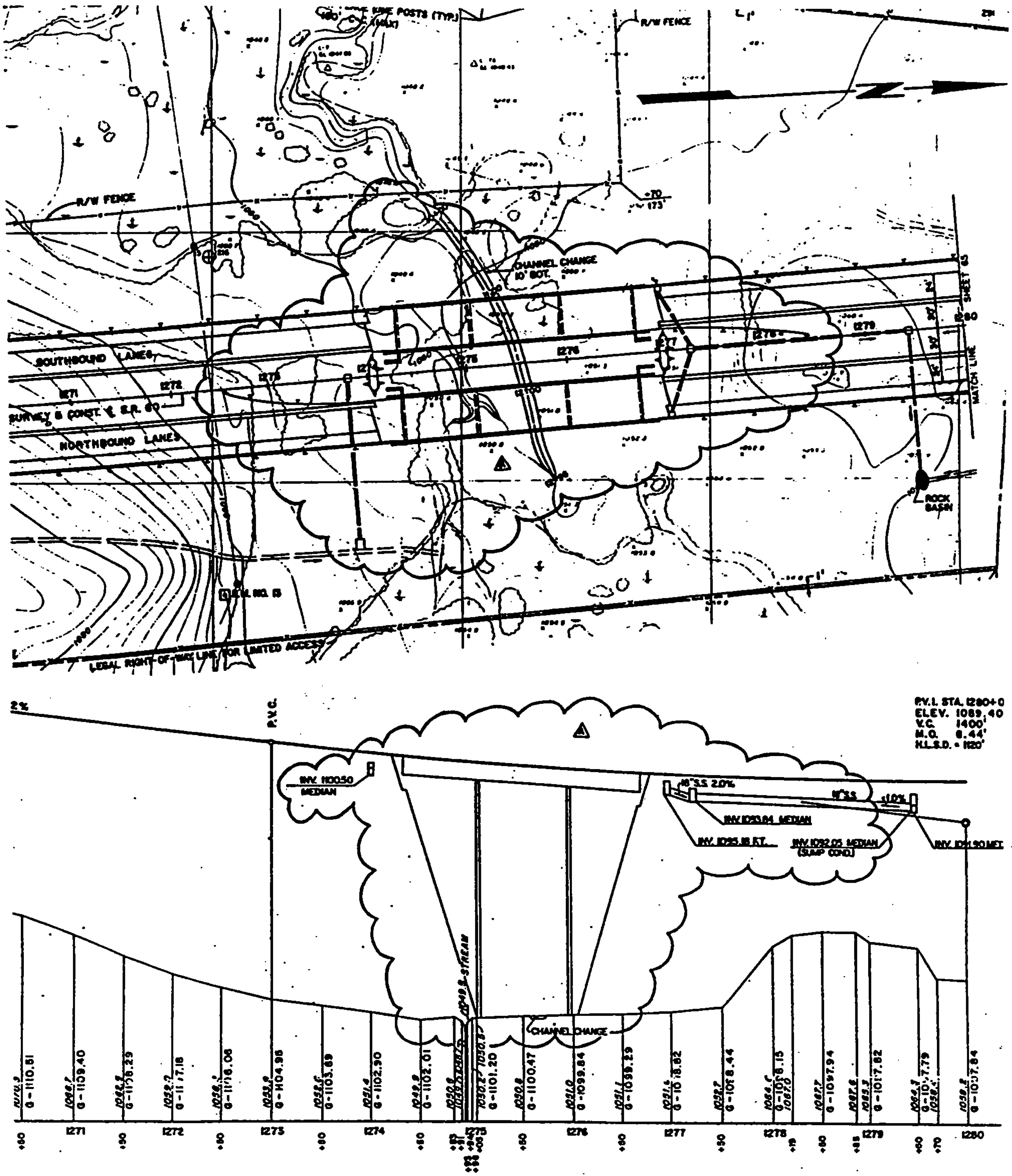
The major geotechnical concerns in this area were stability and consolidation settlement of these embankment foundation soils.

A slope stability analysis was run using the PCSTABL5M program, applying the modified Janbu method. A minimum safety factor of 1.71 was obtained, which exceeded the design requirement of 1.5. However, a five-foot thick sandstone embankment base was specified to further ensure stability and allow free drainage of the foundation soils without saturation of the embankment. In addition, twenty-foot wide by six-foot deep toe benches were to be constructed to remove the upper soft clay layer in the critical toe-of-slope areas.

The settlement analysis relied on consolidation test results from a Shelby Tube pushed in the lower clay stratum and an empirical relation between the standard penetration resistance and the bearing capacity index for the other strata. A maximum settlement of 8.4 inches was calculated. It was calculated that approximately 2.5 years would be required for 90 percent consolidation of the deep clay layer. However, after 35 percent consolidation of this layer, a total of only roughly three inches of settlement would remain. This 35 percent consolidation could be accomplished within 90 days of completion of the embankment. Considering the uniformity of the total foundation soil thickness and the height of the embankment, the resulting differential settlements would be small. Therefore, potential settlement concerns were addressed simply by requiring a minimum of 90 days between construction of the embankment to rough grade and final grading and paving operations.

Environmental concerns are raised

In July of 1989, with the design of the embankment and culvert nearing completion, a meeting was held among the various natural resource agencies, the designers and the PA Turnpike Commission. The intent of the meeting was to review the project designs for stream crossings and wetland impact areas and verify that these designs were acceptable to the agencies. Concern was raised by the PA Game Commission that the embankment and culvert would destroy the integrity of the wetland. The option of constructing a bridge in lieu of the embankment and culvert was discussed. A cost in excess of 3 million dollars was estimated for a dual structure. Both the PA Game Commission and the PA Fish Commission deferred rendering an opinion relative to preferred construction until after a field view of the area.



ADDENDUM PLAN & PROFILE

FIGURE 2

In March of 1990, with the responses of the agencies still outstanding, the Section 42 of the project was advertised for construction. Bid opening would be in early May to allow construction to start in late summer of 1990. However, the permit applications for the project were in jeopardy. The PA Department of Environmental Resources wanted full consideration of the bridge option and requested bridge plans to review prior to issuing a formal response. The Game Commission was concerned about wildlife habitat loss and the Fish Commission was concerned about wetland damage.

On April 20, 1990, the designer was directed to design a dual, three span structure with a 100-foot opening at the wetlands elevation. The preliminary design drawings and estimates were issued May 1,1990, as Addendum No. 3 to the contract documents for construction. See Figure 2. The actual bid price, based on the preliminary design, was 1.58 million dollars. Final plans and estimates were delivered to construction on July 27, 1990, after award of the contract. Payment of construction of the bridge was to be determined by applying unit prices for component items to final measured quantities.

Geotechnical designs are revisited

The bridge structure was designed with continuous steel girders. Allowable deflections are typically limited to one inch with this design. The pier foundations would be H-piles end-bearing on bedrock, while the abutments would be founded on spread footings in the embankment.

The settlement calculations indicated that approximately 28 months would be required to complete 90 percent (all but 0.9 inches) of settlement if no special measures were taken. In order to decrease this time, vertical wick drains were proposed to be installed to the base of the deep clay layer. Embankment settlement could then be monitored to allow construction to proceed only after sufficient settlement had occurred. Special provisions were already in the contract for use of wick drains and surface monitoring of embankments at other locations. Detailed plans were developed to implement the same procedures here, in order that 90 percent of the settlement would be complete 90 days after completion of the embankment to rough grade.

It was noted that the PA Department of Transportation had had problems with abutments founded in high embankments in the western part of the state and that the geotechnical consultant's settlement calculations were unchecked. The required monitoring period was modified from the consultant's recommendation of 90 days to 180 days, as this could be accommodated without adverse construction schedule impact. The monitoring period was split into two phases: a first 90-day period prior to pile driving at the piers, and then another 90 day period prior to work on the abutments. Potential longer term settlement problems would be addressed through modifications to the abutment/approach slab details and incorporation of jacking provisions. In addition, the bridge designer had determined that the continuous girders could handle 3-inch deflections at the abutments.

Construction begins on schedule

The contractor received a notice to proceed with construction on Section 42 on August 20, 1990, and a required completion date of November 20, 1992. By October, the contractor was actively planning the approach embankment work for BV-116. The availability of sandstone from project excavation was questioned. Very little sandstone was anticipated from project excavation. Only 10,000 cubic yards of sandstone borrow had been incorporated in the contract, instead of the 250,000 cubic yards originally recommended. The approach would be to use the best available material on the project, involving geotechnical representatives to evaluate each specific situation. For the approaches to BV-116, it was decided that piles of overburden from the adjacent unreclaimed strip mines would be acceptable sources of material for toe benches and the rock blanket. While not entirely sandstone, these piles were generally granular in nature with cobble and boulder sized sandstone, siltstone and shale in a matrix of sand, silt and clay. The contractor also was required to place a 2-foot lift of granular borrow material on top of the mine spoil in areas of wick drain placement.

The wick drain specialty subcontractor and wick drain materials were identified in October, but actual installation of wick drains at BV-116 was delayed until April, 1991. When the subcontractor arrived on site, it was discovered that as much as ten feet of embankment had been placed over much of the area. The mandrel to be used for wick drain placement could not penetrate the stabilization lift. Even with predrilling through this lift, the drains could not be installed to the originally specified depth of 60 feet below original ground. Due to limitations of the contractor's equipment, depths of only 54 to 56 feet were achievable at best. Wick drain installation was completed prior to May 10, 1991.

Both approach embankments were constructed to rough grade within the next month. Survey monuments were placed as required and initial readings were taken on June 7, 1991. The monuments were surveyed on a weekly basis for the first 12 weeks. As of September 5, 1991, the 90-day period for the pier areas was complete and the contractor was allowed to proceed with pier construction, provided the survey monuments continued to be surveyed on a weekly basis. It was hoped that the survey monument readings would show that the settlement was essentially complete at the end of 90 days. However, the settlement curve was inconclusive at best.

In September and October, 1991, the pier piles and concrete pile caps were placed. On October 17, 1991, the contractor requested a waiver on the 180-day waiting period at the abutments, even though a significant amount of settlement was still occurring. Construction of the piers was completed by Thanksgiving. The Project Manager responded to the waiver request on December 4, 1991 (the date when the 180-day period was completed). In the face of potential delay claims, the contractor was allowed to commence abutment construction, provided survey monitoring was continued and additional survey points were established and monitored on each abutment as soon as it is poured.

A problem starts to develop

In response to the contractor's waiver request, the available technical information was reviewed in detail. This review indicated that:

1. Settlement of the soils had not reached 90 percent even after 180 days. In fact, a regression analysis indicated that the embankment monitoring data could be represented by a straight line on a settlement vs. square root of time graph with a goodness of fit of -0.98 or better.

2. The original calculated settlement of 8.4 inches appeared to underestimate the total settlement. Independent (checked) calculation of the settlement using the original test results, geometry and methodology indicated that settlement would be closer to 47 inches.

3. Due to construction difficulties, the wick drain may not have met the design intent. Based on the independent calculations, settlements to date should have been approximately 34 inches, instead of 5 to 6 inches, if they had worked as designed.

4. Additional settlement may exceed 3 inches. This was a conservative statement to notify the appropriate parties that the design limit of the structure might be exceeded.

As a result of discussions among the Commission, the general consultant, the project manager, the section designer and the designer's geotechnical subconsultant, the contractor was directed to place the southern abutments 2 inches higher than designed and the northern abutments 1 inch higher than designed. However, completion of the bridge was now on the contractor's critical path schedule, and the project manager did not intend to delay the contractor, since this was not perceived as a problem that would seriously impact construction quality.

During January and February, 1992, the contractor proceeded to construct the abutments. The top of the embankment was excavated, the forms were constructed and the concrete poured. However, backfill was not placed behind the abutment. During March, the steel was set. The forming was in place to allow pouring of the concrete deck during the week of April 20, 1992. On April 16, 1992, the contractor surveyed the elevations on the tops of the girders at the centerline of bearing and compared them to the elevations recorded when they were first set (March 4):

Average change at Abutment 1 southbound	-1.38 inches
Average change at Abutment 2 southbound	-2.18 inches
Average change at Abutment 1 northbound	-1.12 inches
Average change at Abutment 2 northbound	-1.78 inches

At an emergency meeting on April 22, 1992, among representatives of the Commission, the general consultant, the project manager, the section designer, and the geotechnical subconsultant, a decision was made to delay placement of the concrete deck. While all parties were in agreement that continued vertical and horizontal movement of the abutments was a serious problem, there was no consensus as to the cause. An additional investigation to determine the cause of the movement and develop a remediation plan was authorized. The investigation, analysis and report was to be completed no later than May 22, 1992.

On May 8, 1992, the contractor submitted a proposal to provide an additional concrete batch plant at a cost of $176,000 to ensure that all work would be completed within the required completion date, with the possible exception of BV-116. It was accepted three days later. The contractor was also notified at the same time of his responsibility to maintain proper documentation for any extra work incurred to complete BV-116.

Investigations get results

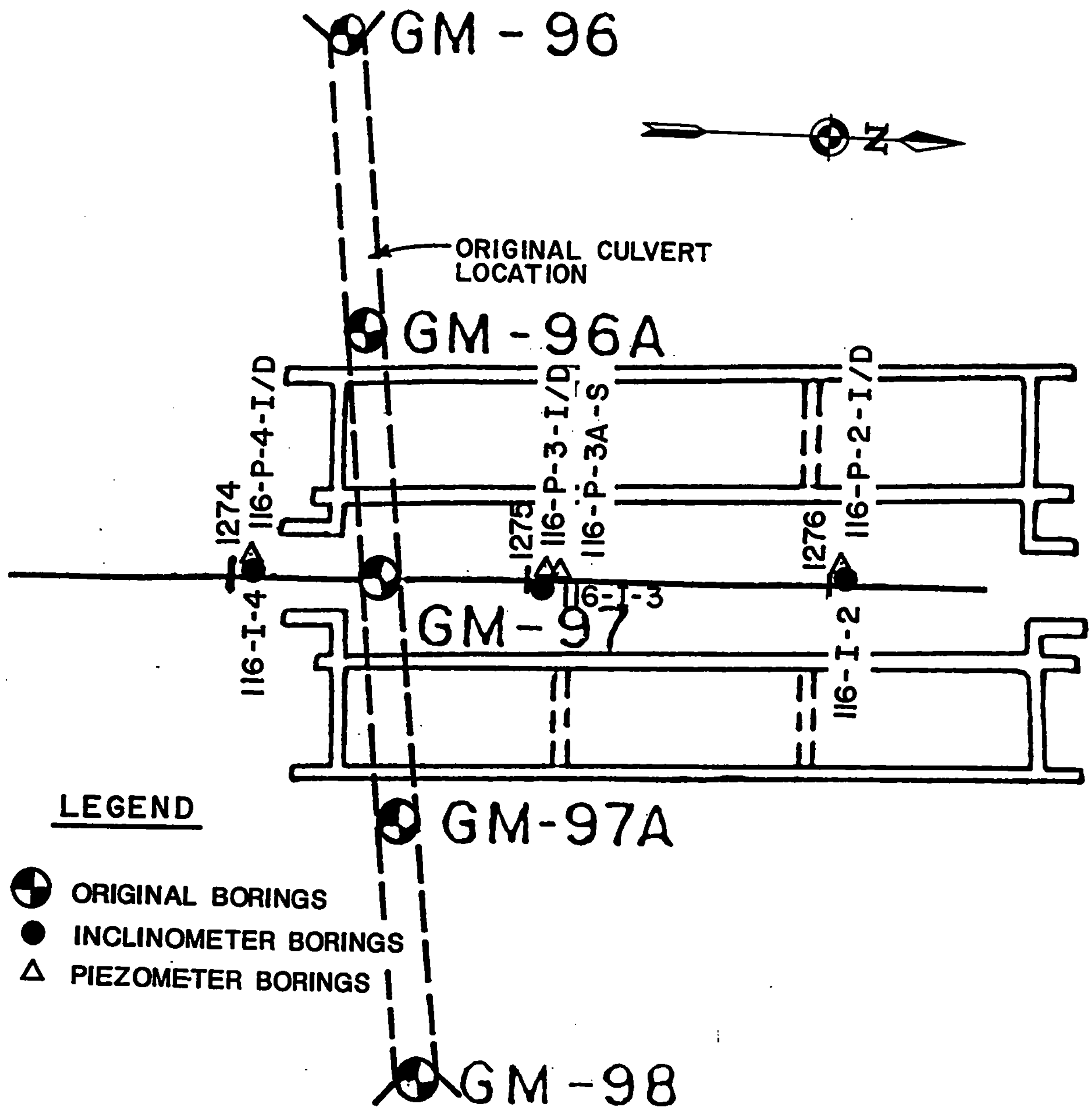
The new investigations included performance of several tasks:

1. Review the existing monitoring data, including surveys of the embankment, the abutments and the girders;
2. Drill eight additional borings to better define subsurface conditions and install slope inclinometers and standpipe piezometers (see Figure 3);
3. Excavate behind the abutments to observe the embankment condition at footing elevation;
4. Test additional disturbed and undisturbed soil samples to refine parameters for analysis;
5. Analyze the settlement and stability of the approach embankments and abutments;
6. Determine the cause of the problem; and
7. Provide recommendations for a solution.

Review of the embankment survey data indicated that only 3 of the original 9 points were still reliable on the south embankment and only 2 of the original 9 points were still reliable on the north embankment. With these points, there still was no apparent break to a straight line interpretation when the data was plotted as settlement vs. the square root of time. See Figure 4. Each abutment apparently had both settled and rotated such that the footing toe was lower than the heel. The movement was significant enough to warrant concern for both deflection and compression of the girders. This concern was supported by the limited girder survey data.

The subsurface drilling program was performed by the Section geotechnical subconsultant's drilling rigs and crews, with continuous inspection and logging by the general consultant's geotechnical personnel. With a series of borings along the profile line of the structure, a more complex subsurface picture came into focus. See Figure 5. A near-surface layer of highly organic silt was found that was not identified in the previous borings. The revised subsurface profile still had a total depth of 70 feet and four layers:

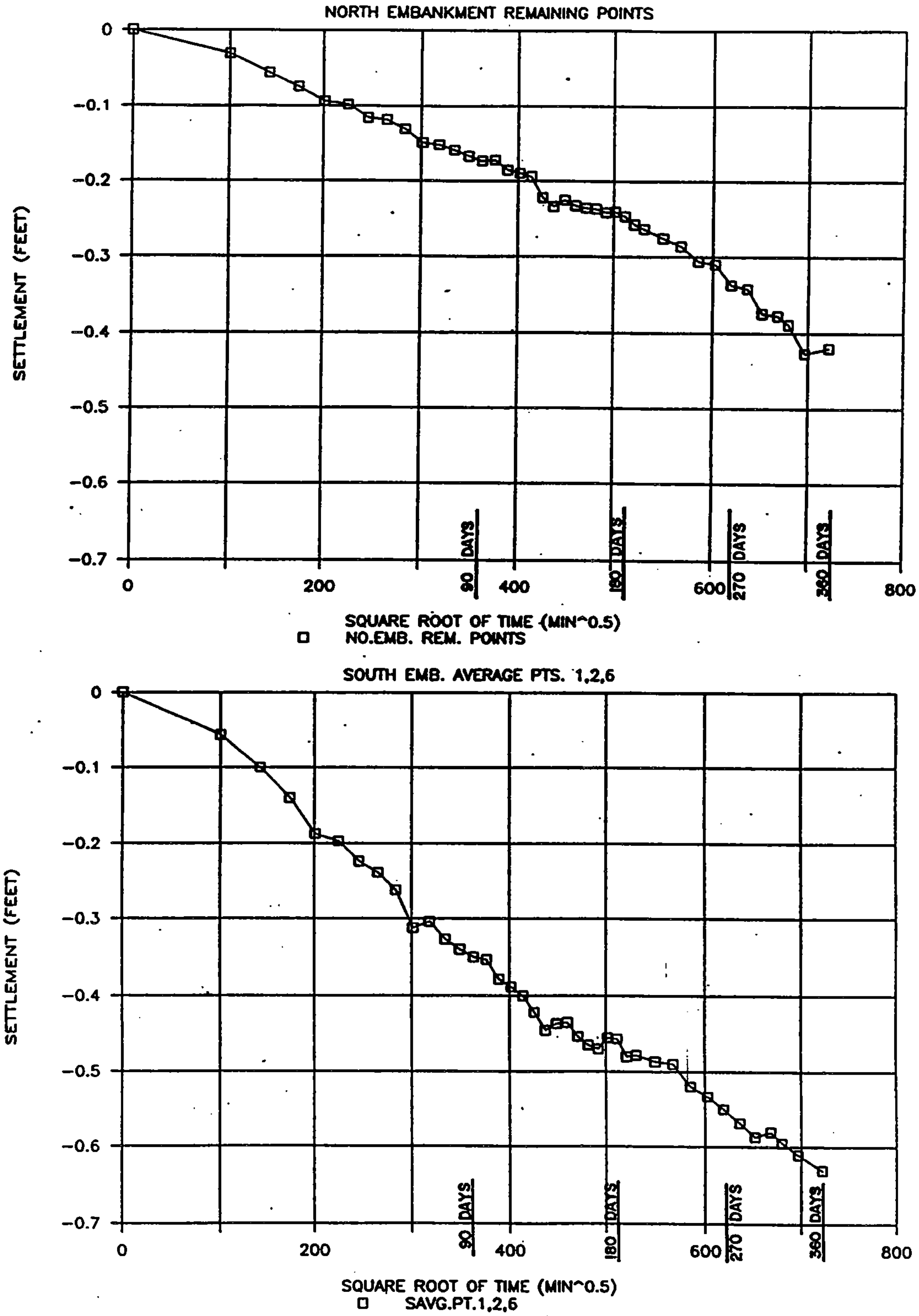
11 feet - highly organic silt, little sand
29 feet - silty sand; gravel; or clay, some gravel - interbedded
18 feet - highly plastic silt or clay
12 feet - rock fragments



REVISED TEST BORING LOCATION PLAN

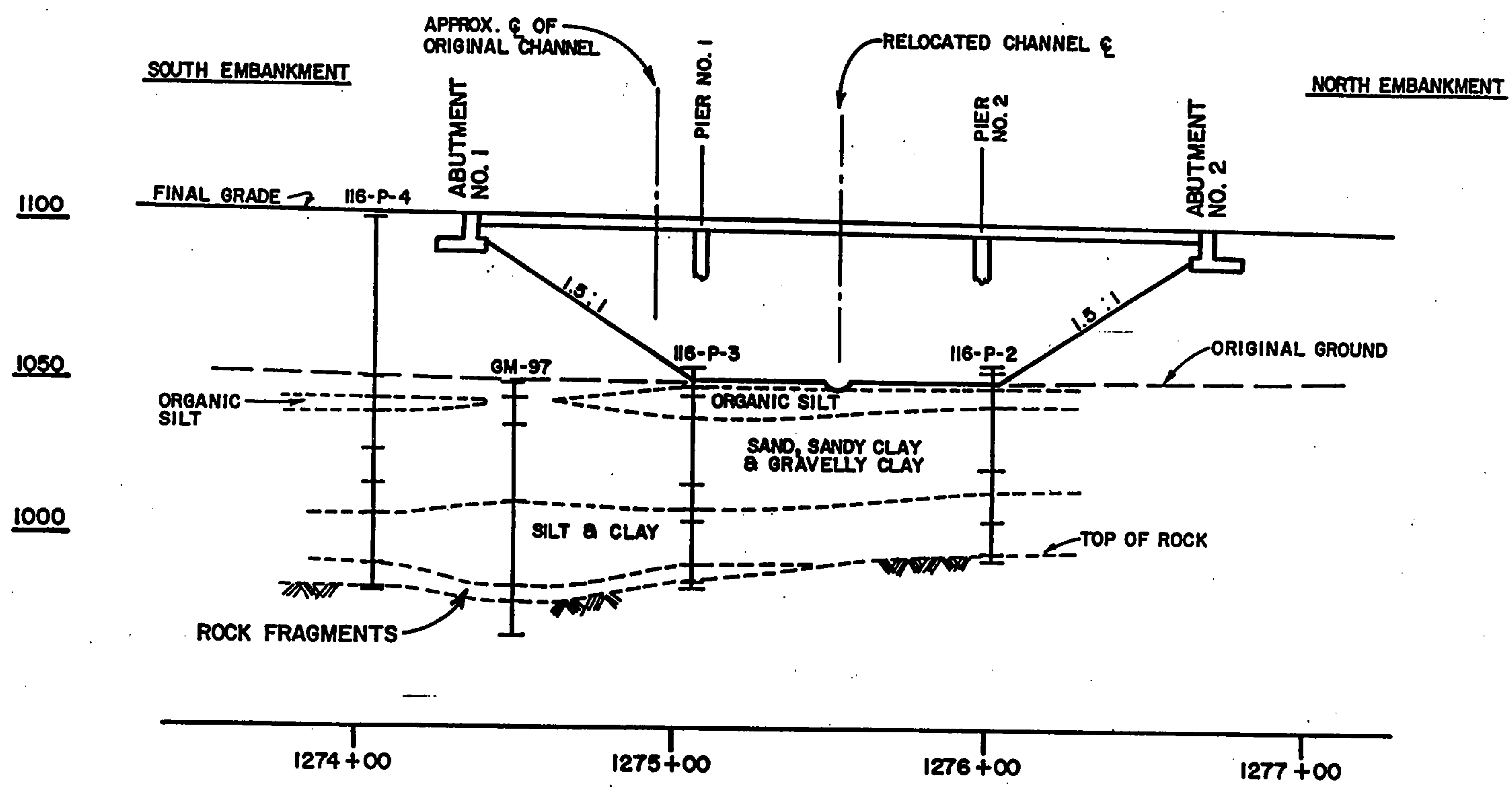
FIGURE 3

BV116 SETTLEMENT MONITORING



EMBANKMENT SETTLEMENT

FIGURE 4



REVISED SUBSURFACE PROFILE

FIGURE 5

The inclinometers and piezometers that were installed did show conclusively any lateral movement or excess pore pressures in any of the soil layers. However, it was noted that the embankment between the abutments and the piers was constructed at a slope of 1.5:1 (H:V). This case had not previously been analyzed for stability; all previous analyses had considered only 2:1 slopes.

It was noticed that since the areas behind the abutments had not been properly backfilled, water had been allowed to pond in these areas since January. The material that had accumulated behind the footings was removed by the contractor in mid-May. Significant water was found behind one abutment. In the other abutment areas, the excavated material and the material at footing elevation was wet. The contractor was immediately directed to take steps to remove all the wet material and take steps to keep water from infiltrating the footing area.

The soil samples collected during the subsurface drilling program were shared between the Section geotechnical firm and the general consultant for independent testing. Given the limited time frame, the amount of testing performed was extensive:

Natural moisture content tests	83
Classification tests	27
Organic content tests	5
Standard consolidation tests	3
Modified consolidation tests	3
Consolidated-drained direct shear tests	2
Consolidated-undrained triaxial compression tests	1

The independent testing marked the start of independent analysis and development of recommendations. The three primary parties involved; the Commission, the Section geotechnical subcontractor, and the general consultant; each focused on different probable causes: softening of the immediate footing subgrade, softening and collapse of the embankment materials, and response of the embankment foundation soils, respectively.

<u>Changes are required</u>

Although the reports were not finalized, a meeting was held on May 21, 1992, to present the findings to date and discuss remediation options. Each of the three parties presented reasons why they felt the cause they were focusing on was a legitimate concern. After considerable discussion, there was no clear consensus as to the extent of the problem. Localized settlement at the abutment footing subgrade, continued consolidation of the embankment foundation soils, and potential instability of the abutment slopes were shared concerns of the Commission and the general consultant. The project manager noted that direction was needed before July 1, 1992, if there was to be any hope of avoiding a delay claim by the contractor. The meeting was adjourned with the charge that the slope stability issue be resolved within one week, and a proposal for structural retrofit at the abutments and geotechnical enhancements be submitted within two weeks.

Slope stability analyses indicated that the existing embankment had a safety factor of only 0.8 (at worst) to 1.1 (at best) against failure under the rock toe bench and through the organic silt. With a buttress, this latter safety factor could be increased to 1.35. Therefore, a buttress definitely was warranted at the toe of the abutment slopes. The effect of the buttress on the piers was analyzed structurally. A buttress design was developed by the project manager to meet the criteria imposed by both slope and pier considerations. In mid June, both the Corps of Engineers and the PA DER were notified that permit revisions were necessary for the project because the buttress was required. The addition of the buttress would increase the total wetland loss to essentially match that of the original permit application with the box culvert.

Settlement analyses based on the new data indicated that combined consolidation settlement of the foundation soils and compression of the embankment soils could be expected to be in the range of 12 to 18 inches. The total measured settlement to date was approximately 8 inches. Therefore, an additional 4 to

10 inches could be expected, with 2 to 8 inches occurring short term and another 2 inches occurring long term. There would be a reasonable chance that additional settlement would be around 6 inches.

The Section designer transmitted their structural and geotechnical recommendations on June 2, 1992. The structural recommendations included substantial reconstruction of the bearing and diaphragm areas to set the girders properly at the design elevations. The modifications would allow the bridge to accommodate 3 inches of additional settlement at the abutments. The provision of shims at the piers would allow an additional 3 inches of settlement. The geotechnical subcontractor recommended compaction and penetration grouting 15 to 20 feet of the embankment material beneath the abutments.

During the month of June, discussions were held the Commission, the general consultant, the project manager, the Section designer, and the Section geotechnical subconsultant to arrive at recommendations acceptable to all parties. The recommendations finally implemented included:

1. reconstructing the abutments essentially as proposed by the section designer, only setting the girders one to two inches high to accommodate more future settlement;
2. installing the buttress prior to pouring the concrete deck;
3. installing horizontal geogrid layers within the structural backfill behind the abutments;
4. continue monitoring the survey points, inclinometers and piezometers; and
5. paving the embankment roadway with bituminous concrete for at least 300 feet from each approach slab.

<u>Construction proceeds as the permit agencies respond</u>

On June 26, 1992, the Corps of Engineers approved the request to place the buttress. On June 30, the contractor was directed to proceed with construction in accordance with the new recommendations. When the PA DER had not responded to the permit revision request by August 13, 1992, the project manager alerted the Commission that progress was once again in jeopardy. The Commission contacted the PA DER and discovered that they were planning to deny the revision request. The PA DER once again wanted to see alternatives, this time for stability remediation.

Three concepts were considered: a rock buttress modified to include a 40-foot opening width, a concrete slope facing/ soil anchor system, and a compaction grouting program for the embankment foundation. The slope facing/ anchor system would have drastically interfered with the wick drain system already in place. The compaction grouting was estimated to cost roughly three times the rock buttress ($320,000 vs. $130,000).

On August 19, 1992, the contractor informed the project manager that he was once again prepared to pour the concrete decks, and would be delayed unless the buttress could be built immediately. Instead, the contractor was allowed to proceed with the deck pours prior to buttress placement, with daily monitoring of the survey points and inclinometers, and a geotechnical representative on the site.

On August 26, 1992, representatives of the Commission and the PA DER met to discuss slope stabilization at BV-116. They reached verbal agreement to allow construction of the modified rock buttress. Formal application and approval were still required; the letter request was sent on September 1, 1992, and the approval was sent on September 9, 1992.

The contractor was notified of the buttress modifications on September 1, and was directed to prepare for its construction upon receipt of approval from the PA DER. The concrete decks and parapets were poured and the rock buttress was placed in September, 1992. The bituminous concrete pavement was placed early in November, 1992. The project manager recommended that Section 42 of the Beaver Valley Expressway be accepted as complete as of November 13, 1992, one week ahead of the contract completion date. The original total contract cost of $31.4 million was exceeded by slightly more than $1.2 million.

Post - construction monitoring continues

At completion of the construction contract, the project manager assumed responsibility for continued monitoring of BV-116 until June of 1993. Since that time, this task has been the responsibility of the Commission.

The embankment monitoring points were effectively destroyed during final grading operations in October, 1992. New monitoring points were established, but have been essentially abandoned. A noticeable depression developed across the pavement behind one abutment, which required a small bituminous overlay.

The inclinometers were read in October, 1992, and April, 1993, for comparison with the May, 1992, readings. While one inclinometer apparently showed movement in excess of one inch, there were difficulties with the readings attributable to misalignment of the casing as it was extended upward through the buttress. Optical surveys of the top of inclinometer casing and field reconnaissance have not revealed any lateral movement at or near the embankment toe.

Monitoring of the abutment itself has indicated that three inches of settlement (after the deck pour - original design criterion) was reached at one abutment by the end of September, 1993. The Commission's bridge unit reanalyzed the structure and determined that it could accommodate up to 5.5 inches prior to pulling shims at the piers and resetting the girders. Current estimates are that this revised criterion may not be reached at two abutments. At the other two abutments, it may be reached as early as 1995. Monitoring of the abutments continues. See Figure 6.

Lessons are learned

The bridge and approaches have continued to carry traffic successfully since opening of the expressway on November 20, 1992. Given the constraints placed on their design and construction, that is quite an accomplishment. However, on reflection, it is through projects like BV-116 that new lessons are learned and old lessons are brought once again to mind. Following are a few of those lessons for consideration:

1. Environmental studies should not be performed in isolation from engineering and design. Input is required from appropriate disciplines (especially geotechnical) in order to effectively determine when impacts are minimized "to the extent possible".

2. Regulations not withstanding, minimum wetland disturbance at any particular site is not necessarily the best option - the best value for the investment.

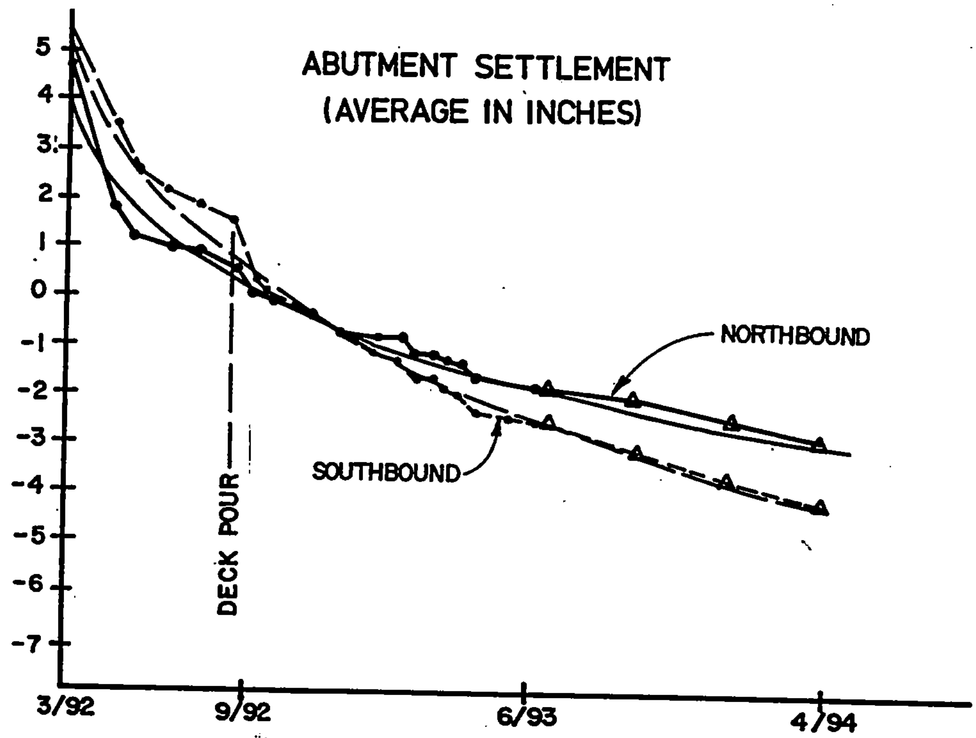
3. Meetings are best with short, focused agendas.

4. Geotechnical investigations should match the design. If the design changes, a new or modified investigation should be considered immediately.

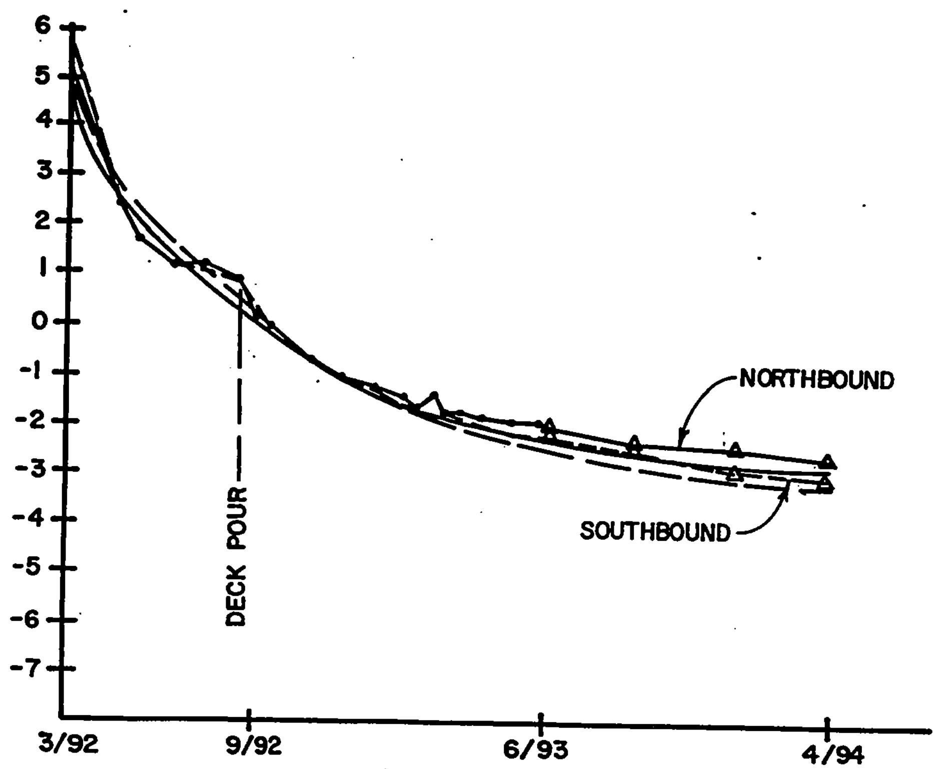
5. Calculations should always be checked, regardless of who performs them initially.

6. The results of the calculations should be reviewed via alternate resources and/or methods, and may need to be tempered with practical knowledge and experience.

7. Geotechnical designs and recommendations should be incorporated properly into the plans and specifications for construction, with this incorporation assured by review of the construction documents by a geotechnical engineer and/or geologist as appropriate.



ABUTMENT NO. 1



ABUTMENT NO. 2

CURRENT SETTLEMENT CURVES

FIGURE 6

8. Adequate instrumentation and monitoring should be included for all geotechnical concerns during construction and beyond. There is no substitute for accurate, site specific data.

9. All provisions of construction contracts should be enforced, regardless of their apparent immediate usefulness.

10. Review and oversight should not stop after design or even after actual construction, but should continue through preparation of construction documentation and post-construction performance of the completed project.

Bibliography

Buchart-Horn, Inc., 1990, Contract Documents and Drawings for Construction of Beaver Valley Expressway, State Route 60, Section 42, PA Turnpike Commission, Harrisburg, PA

DeWolf, F.W., 1929, Topographic and Geologic Atlas of Pennsylvania, No. 5, New Castle Quadrangle, Pennsylvania Geological Survey, Harrisburg, PA

Gannett Fleming Transportation Engineers, Inc., 1989, Environmental Assessment, Beaver Valley Expressway, Beaver and Lawrence Counties, Pennsylvania, PA Turnpike Commission, Harrisburg, PA

Gannett Fleming Transportation Engineers, Inc., 1989, Composite Technical Basis Report, Beaver Valley Expressway, Beaver and Lawrence Counties, Pennsylvania, PA Turnpike Commission, Harrisburg, PA

GeoMechanics, Inc., 1989, Report of Final Soils and Geologic Engineering Investigation, Beaver Valley Expressway, PA Turnpike Commission, Harrisburg, PA

GeoMechanics, Inc., 1992, Report of Geotechnical Investigation, Bridge BV-116, Beaver Valley Expressway, PA Turnpike Commission, Harrisburg, PA

USDA/Soil Conservation Service, 1982, Soil Survey of Beaver and Lawrence Counties, Pennsylvania, US Department of Agriculture, Washington, DC

PAVEMENT ANALYSIS USING ARKPAV

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Abstract

The computer program ARKPAV is presented for pavement analysis using nonlinear finite element. It is similar to ILLI-PAVE in modelling the material behavior of pavements. ARKPAV uses an efficient and compact storage scheme and economical solution procedure based on Incomplete Cholesky Conjugate Gradient(ICCG) procedure. ARKPAV analyze a pavement on a 16x17 mesh in 33 seconds compared to 468 seconds by ILLI-PAVE in a 386/33mhz microcomputer. Also the storage space is reduced by 8 times compared to banded solution procedure (ILLI-PAVE) for a 16x17 mesh. This ratio increase as the bandwidth increases. The performance of the two programs are compared with ELSYM5 for linear analysis. Results are also compared for a sample nonlinear pavement analysis. The computed results are in good agreement.

INTRODUCTION

The computer program ILLI-PAVE (1982) is perhaps the most powerful and realistic tool presently available for the structural analysis of flexible pavements. This program, developed at the University of Illinois, models the nonlinear, stress dependent behavior of unbound granular base and subgrade soils. It also includes a stress adjustment feature based on the Mohr-Columb failure theory to compensate for stress states that exceed the strength of the materials.

ILLI-PAVE is the culmination of the efforts of several researchers over many years. As a result it is long and quite complex. It is based on older FORTRAN compilers and uses data input routines that are quite cumbersome by todays standard. Its output is voluminous. This combination makes the program difficult to modify for further research work and the voluminous output hinders ease of use for more routine purposes.

To rectify some of the above mentioned problems; the computer program ARKPAV was developed and initial developments were reported in Selvam and Elliott (1992). The program is rewritten to make it user friendly. The simple program structure of ARKPAV makes it easier for future modifications like dynamic analysis of pavements or to incorporate new material models. The four noded isoparametric quadrilateral finite element used in ARKPAV can compute the displacements and stresses with much better accuracy than the quadrilateral element based on four constant strain triangles used in ILLI-PAVE.

With the advancement in microcomputer hardware and software, ARKPAV is developed to run on microcomputers. First of all to reduce the amount of storage space needed to store the equations formed using finite element procedure, a compact storage scheme and an iterative procedure is implemented. This reduces the storage space by 8 times compared to semi bandwidth storage scheme for a 16x17 mesh. This ratio increases as the semi bandwidth of the mesh is increased. The equations are solved using efficient incomplete Cholesky conjugate procedure (Selvam, 1994). For a 16x17 mesh sample problem, ARKPAV took 33 seconds and ILLI-PAVE took 468 seconds in the same microcomputer. Hence the program is very much suitable for every day pavement analysis.

Currently work is underway to prepare input using a user friendly program before running ARKPAV. In this work the details of ARKPAV and its merits are discussed with illustrations.

MATERIAL BEHAVIOR MODELING

A typical flexible pavement consists of three or more layers as depicted in Fig. 1. With the brief loading times associated with moving vehicle loads, the asphalt concrete surface layer behaves in a manner that is essentially elastic. This material can be reasonably modeled as linear elastic. However, the other two layers behave in a non-linear, stress dependent fashion that complicates the mathematical modeling.

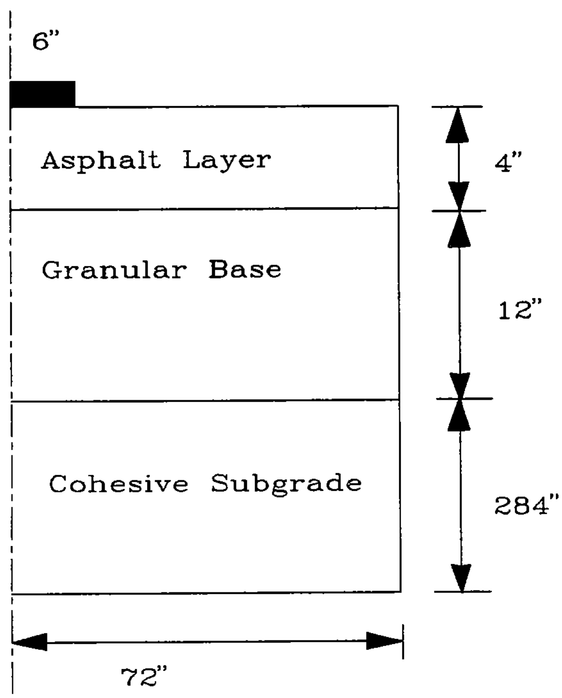

Fig. 1. Flexible Pavement Considered for Analysis.

The granular base material behaves in "stress stiffening" manner and the cohesive subgrade soil behaves in a "stress softening" manner. For granular soils

$$M_R = k(\theta)^n \quad (1)$$

in which M_R is the resilient modulus; $\theta = \sigma_1+\sigma_2+\sigma_3$; σ_1, σ_2,σ_3 = principal stresses; and k,n= material constants. For subgrade soils

$$M_R = K_2+K_3[K_1-(\sigma_1-\sigma_3)]; \text{ when } K_1 > \sigma_1-\sigma_3 \quad (2a)$$

$$M_R = K_2+K_4[(\sigma_1-\sigma_3)-K_1]; \text{ when } K_1 < \sigma_1-\sigma_3 \quad (2b)$$

in which K_1,K_2,K_3,and K_4 = material constants. Models for these behavior battens were incorporated into ILLI-PAVE by Duncan et al. (1968).

For many pavement systems, these models alone were not sufficiently complete since the calculated stress states often exceeded the strength of the materials. This was particularly true for the unbound, granular base and subbase material as well as the upper portions of the subgrade. Raad and Figueroa (1980) improved the ILLI-PAVE model by incorporating a stress adjustment feature based on Mohr-Coulomb failure theory. ARKPAV incorporated these material models in a structured form suitable to include new models in the future.

Another advantage of ARKPAV is that the programming is simpler and more straight forward. ILLI-PAVE was developed over many years with many researchers contributing to todays program. As a result, the programming is complex and not fully documented. Analysis of the ILLI-PAVE code for de-bugging when installing on a different machine or for the incorporation of additional modifications is quite time consuming. The simpler structure of ARKPAV will make the incorporation of any future modification much easier. With all of the modifications, the required storage for the uncompiled FORTRAN code (source code) for ARKPAV is about 25 k compared to 360 k for ILLI-PAVE.

COMPUTER MODELING

The flexible pavement having the nonlinear behavior mentioned in the previous section is analyzed for stresses and strains using finite element method. ILLI-PAVE models the three-dimensional pavement as an axisymmetric solid of revolution. The pavement region is divided into quadrilateral elements formed out of four triangular elements. The triangular element used in ILLI-PAVE was incorporated by Duncan et. al. in 1968. Currently isoparametric quadrilateral elements which can predict displacements and stresses with much better accuracy than triangular elements are available (Bathe 1982) in finite element literature.

ARKPAV was developed incorporating the nonlinear material models used in ILLI-PAVE but using the more accurate isoparametric quadrilateral element. ARKPAV is written in FORTRAN77 which is compatible with most computers. The input preparation is free format and very simple to prepare.

ILLI-PAVE prints the stresses and strains of each element and the displacements of each node for each iteration. The output is rather voluminous. In many cases, the pavement analyst is only concerned with the radial strain at the bottom of the asphalt concrete layer and the vertical strain at the top of subgrade. ARKPAV includes an option to print only this information.

SOLUTION PROCEDURE

In the ILLI-PAVE program the resulting symmetric simultaneous equations from finite element method are stored in a banded matrix form considering only the upper triangular part. These are solved by a direct solution procedure similar to Cholesky decomposition. Instead of solving the banded matrix in core, the matrix is divided into blocks and only two blocks are considered at one time in core. The rest of the blocks are stored in out of core devices. By this process large set of equations can be solved with the available storage. The minimum size of the block should be at least twice as large as the semiband width.

In a microcomputer, the time to transfer from disk to core takes a relatively long time. To eliminate the need for this transfer, a more efficient storage scheme is used in ARKPAV. This scheme stores only the non-zero elements of the upper triangular matrix in an array illustrated as A in Fig.2 (Axelsson and Barker, 1984; Johnson, 1987 and Selvam, 1994). As illustrated, the values are stored in order by row. The column locations of the non-zero values are stored in another array labelled LC. A third array, LR, is used to identify the row locations. In array LR, the ith value identifies the beginning point in A of the ith row of the matrix. For example in Fig. 2., the third value in LR is 7 which indicates that the third row of the matrix starts from the 7th element of A. This storage scheme is independent of the bandwidth which greatly reduces storage needs. For pavement analysis using quadrilateral element maximum ten storage spaces are needed for each equation. For example, the storage needed for a 16x17 and 23x42 mesh in ARKPAV is about 22k or 5,440 storage spaces and 77k or 19,320 storage spaces respectively. For the same meshes using the banded matrix method used in ILLI-PAVE requires 157k or 19,584 storage spaces and 773k or 96,600 storage spaces respectively.

$$
A = \begin{bmatrix}
a11 & a12 & 0 & a14 & a15 \\
 & a22 & a23 & 0 & 0 \\
 & & a33 & a34 & a35 \\
 & \text{symm} & & a44 & 0 \\
 & & & & a55
\end{bmatrix}
$$

	1	2	3	4	5	6	7	8	9	10	11	
A	[a11	a12	a14	a15	a22	a23	a33	a34	a35	a44	a55]	
LC	[1	2	4	5	2	3	3	4	5	4	5]	
LR	[1				5		7			10	11	12]

Fig. 2 Illustration of the Compact Storage Scheme

ARKPAV further reduces storage requirements by solving the equations using an iterative solution procedure. The direct procedure used by ILLI-PAVE requires double precision for acceptable accuracy. With the iterative procedure, single precision provide acceptable accuracy. The equations are solved by preconditioned conjugate gradient iterative procedure. There are different preconditioning procedures are available in the literature (Axelsson and Barker, 1984; Mitchell and Griffiths, 1980; Oden and Carey, 1982 and Selvam, 1984). In this work Incomplete Cholesky Conjugate Gradient(ICCG) procedure is used. The details of the storage scheme and the implementation of ICCG procedure is given in Selvam (1994). The following is the ICCG algorithm for simultaneous equations of the type Ax=b (where A is a symmetric matrix and x and b are unknown and known vectors):

x_0 = starting guess at solution
$r_1 = b - Ax_0$
$p_0 = 0$ and $r_0 M^{-1} r_0 = 10^{20}$
For k=0,1,2,... do until required convergence
$\beta = r^t_{k+1} M^{-1} r_{k+1} / (r^t_k M^{-1} r_k)$
$p_{k+1} = M^{-1} r_{k+1} + \beta p_k$
$\alpha = r^t_{k+1} M^{-1} r_{k+1} / (p^t_{k+1} A p_{k+1})$
$x_{k+1} = x_k + \alpha p_{k+1}$
$r_{k+2} = r_{k+1} - \alpha A p_{k+1}$

In this r_k, x_k, p_k, b are vectors and α and β are scalars. Here M is the preconditioned matrix. The proper choice of M is essential for faster convergence and to be suitable for practical applications. Here M is formed by incomplete Cholesky decomposition on A. The above iteration is performed until the required convergence is reached. This step of solving Ax=b until required convergence is called outer iteration. This outer iteration is done until the updated resilient modulus of the materials do not have much difference from the previous outer iteration. When performing the nonlinear outer iteration, the solution of Ax=b for the first iteration takes little bit more iteration because of assumed initial x is for linear solution. The second and subsequent outer iterations take less number of iteration in ICCG because x is very close to the actual value. Usually four outer iteration are enough for convergence.

In ILLI-PAVE the equations of the type Ax=b are solved using block by block banded Gaussian elimination procedure. By this the matrix A is split into blocks which has to have a minimum columns and rows equal to width of bandwidth (NB) and 2*NB respectively. By this the required storage space for the program is reduced but transferring the data in and out of the disc consumes more computer time. A program called ARKPAV6 is also developed in this regard for comparison. To solve 16x17 and 23x42 mesh using ARKPAV6, the details of storage spaces using ARKPAV6 are NBLOCK= 3 and 13, NB= 36 and 50 and NEQB=208 and 150 respectively. Here NBLOCK is the number of blocks used, NB is the semi bandwidth and NEQB is the number of equations per block. Program ARKPAV and ARKPAV6 are compiled using the recent WATCOM-FORTRAN compiler called WATCOM-F32. Computer time to run these two sample meshes in a 386/33mhz microcomputer is reported in Table 1.

Program	Mesh(storage space)	cpu(sec.)
ARKPAV	16x17 (22k) 23x42 (77k)	33 48
ARKPAV6	16x17 (157k) 23x42 (773k)	59 336
ILLI-PAVE	16x17 (157k)	468

Table 1. Performance of Different Programs

The ratio of cpu for ARKPAV6 to ARKPAV is more for 23x42 mesh than 16x17 mesh. The reason is the more the number of unknowns to be solved and NB, the more the ratio becomes. In addition the transfer of data in and out of the disk also increased when the NBLOCK is increased. Hence banded solution procedure is not attractive when the problem size is increased. Especially for three-dimensional problems ICCG is very efficient and economical than banded solution procedure. This is clearly demonstrated in Selvam (1994). ARKPAV6 took 59 sec compared to 428 sec by ILLI-PAVE even though same solution procedure is used. This is due to the type of FORTRAN compiler used and may be the structure of the program. The same ARKPAV for a 16x17 mesh took 260 sec using older WATCOM compiler compared to 33 sec using newer WATCOM. Hence the developments in hardware and software helps to accelerate the solution procedure very much. In Selvam and Elliott (1992) the same 16x17 mesh is solved using Jacobi Conjugate Gradient(JCG) procedure and took 420 sec using older WATCOM compiler.

RESULTS

First to study the performance of ARKPAV using isoparametric quadrilateral element a linear analysis is performed using 16x17 mesh for a typical pavement shown in Fig. 1. The layer thickness are 4, 12 and 284 inches; the modulus of elasticity are 500000, 4000, and 3000 psi; and the Poisson's ratio are 0.67, 0.6, and 0.82 respectively for asphalt, base and subgrade. The computed deflection at the top of asphalt, radial strain at the bottom of asphalt and vertical strain at the top of subgrade are compared to the closed form solution computed using ELSYM5 (Kopperman, 1986) in Table 2. For the same mesh the computed results using ILLI-PAVE are also reported in Table 2.

Next a nonlinear finite element analyses was performed using the same mesh size of 16x17 for a typical pavement shown in Fig. 1 to compare different design parameters from ILLI-PAVE to ARKPAV. The failure modulus values for asphalt, base and subgrade are taken as 500000, 4000 and 3000 psi respectively. The layer thickness are 4, 12, and 284 inches for asphalt, base and subgrade layers respectively. The earth pressure at rest are 0.67, 0.6, and 0.82; the Poisson's ratios are 0.4, 0.38 and 0.45; and the densities are 145, 135 and 120 pcf respectively for asphalt, base and subgrade. The material coefficient k and the exponent n for the granular material are 5000 psi and 0.5 respectively. The maximum allowable stress ratio and the minimum horizontal compressive stress (psi) that the granular material is permitted to reach before tensile failure are 4.8 and 0.01 respectively. For the cohesive subgrade the deviator stress(K_1 in Eq.2) and the corresponding modulus value at the turning point of the deviator stress resilient modulus relation(K_2 in Eq. 2) are 6 and 6000 psi respectively. The slopes of the left and right portions of the deviator resilient modulus relation are 1000 and -200 respectively. The lower and upper limiting values for deviator stress in the deviator stress resilient modulus relation K_3 and K_4 in Eq. 2 are 2 and 21 psi respectively. The shear strength beyond which failure occurs is considered as 15 psi. The tire pressure is assumed to be 80 psi and applied on a radius of 6 inches. This is the data set used in ILLI-PAVE (1982) users manual. The nonlinear relationships are well explained in Raj (1991) and ILLI-PAVE(1982).

Program	ELSYM5	ARKPAV	ILLI-PAVE
Vertical Deflection (in)	0.0263	0.0213 (19%)	0.0208 (20.9%)
Radial Strain on Base	0.331×10^{-3}	0.289×10^{-3} (12.7%)	0.276×10^{-3} (16.7%)
Vertical Strain on Subgrade	0.628×10^{-3}	0.487×10^{-3} (22.5%)	0.469×10^{-3} (25.3%)

Table 2. Comparison of Results for Linear Analysis

Using the above input parameters the deflection at the top of asphalt, radial strain at the bottom of asphalt and vertical strain at the top of subgrade are considered for comparison. These parameters are reported in Table 3. The displacement and strains are almost the same value from ARKPAV and ILLI-PAVE. The displacement and radial strain from ARKPAV are slightly higher than ILLI-PAVE. Further details on mesh refinements and its effects on the improvement on the displacement and strains are reported in Raj(1991).

Program	ARKPAV	ILLI-PAVE
Vertical Deflection	0.02380 in.	0.02303 in.
Radial Strain on Base	0.3040×10^{-3}	0.2804×10^{-3}
Vertical Strain on Subgrade	0.5781×10^{-3}	0.5813×10^{-3}

Table 3. Comparison of Results from ARKPAV and ILLI-PAVE for Non-Linear Analysis.

CONCLUSIONS

A nonlinear pavement analysis program ARKPAV which is similar to ILLI-PAVE material model is developed. ARKPAV uses efficient isoparametric quadrilateral element. The program is made usable in microcomputer by storing them in a compact scheme. For a 16x17 mesh about 8 times storage saving is achieved using ARKPAV than ILLI-PAVE. The saving in storage increases with increase in the mesh size. The equations are solved using Incomplete Cholesky Conjugate Gradient iterative procedure which is much efficient than direct solution procedure for large problems. In IBM386/33mhz equivalent machines a 16x17 mesh takes about 33 seconds using ARKPAV and 468 seconds using ILLI-PAVE. The results from ARKPAV and ILLI-PAVE for a sample problem is in good agreement. This program is easy to modify for future use and it is very much suitable for pavement analysis in day today use. Further work is underway to prepare standard mesh and other inputs using another program before running ARKPAV. It is available for anyone to uses it for a reasonable cost.

ACKNOWLEDGEMENTS

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REFERENCES

Axelsson, O., and Barker, V.A.,(1984),**Finite Element Solution of Boundary Value Problems**: Theory and Computation, Academic Press, Inc. New York.

Bathe, K.J.,(1982), **Finite Element Procedures in Engineering Analysis**, Prentice Hall.

Carey, G.F.,and Oden, J.T., (1984), **Finite Elements: Computational Aspects**, Vol. 3, Prentice-Hall, New Jersey.

Duncan,J.M, Monismith, C.L., and Wilson, E.L., (1968), "Finite Element Analysis of Pavements," **Highway Research Record** No. 228, Highway Research Board, Washington, D.C., pp.18-33.

ILLI-PAVE Users Manual (1982), Transportation Facilities Group, Department of Civil Engineering, University of Illinois at Urbana- Champaign, May

Johnson, C., (1987),**Numerical Solution of Partial Differential Equations by the Finite Element Method**, Cambridge University Press, New York.

Kopperman, S. et. al. (1986),"ELSYM5 interactive Microcomputer version, Users Manual: IBM-PC and compatable version," Report number: FHWA-TS-87-206, Federal Highway Administration.

Mitchell, A.R., and Griffiths, D.F., (1980),**The Finite Difference Method in Partial Differential Equations**, John Wiley & sons, New York.

Radd, L., and Figueroa, J.L., (1980),"Load Response of Transportation Support Systems," **Transportation Engineering Journal**, ASCE, Vol. 106, No. TE1, pp. 111-128.

Raj, K., (1991),"Nonlinear Finite Element Analysis of Pavements," MSCE thesis, Department of Civil Engineering, University of Arkansas, Fayetteville, AR 72701, 77 pages, August.

Selvam, R.P. and Elliott, R.P., (1992),"Nonlinear Finite Element Analysis of Pavements Using Microcomputer," **Proceedings**: 43rd Annual Highway Geology Symposium, Fayettville, Arkansas, August 19-21, pp. 273-279.

Selvam, R.P., (1994),"Implementation of Preconditioned Conjugate Gradient Solution Procedure for Large Structural or Finite Element Problems," Report, Department of Civil Engineering, university of Arkansas, Fayetteville, AR 72701, also to be submitted for **ASCE J. Computing in Civil Engineering**.

THE IMPACT OF REGISTRATION OF GEOLOGISTS ON THE PROFESSIONAL INVOLVED IN HIGHWAY GEOLOGY

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ABSTRACT

As of July 1, 1994, 17 states have laws requiring registration of geologists. The requirements vary somewhat from state to state. Another three states require certification, and four others have laws defining "geologist". There are 13 additional states which are in the process of preparing legislation, with several of these introducing legislation in 1994.

The National Association of State Boards of Geology (ASBOG) is a viable organization with a present membership of nine states. An examination has been prepared by this organization with the purpose of standardizing geologist examinations similar to those of public accountants, architects, etc. There are four states who have adopted this exam for geologists at present. This organization is also studying reciprocity, continuing education, and minimum requirements.

Since the primary purpose of this registration is for the protection of the public, anyone practicing in the field of highway geology is directly impacted. Registration improves the status of the geologist working in the highway geology field, but also emphasizes their responsibilities. They should now be directly involved in determining whether the engineer or the geologist is responsible for certain tasks. It is in the best interest of the public for the professional to actively participate in maintaining and promoting registration.

INTRODUCTION

The purpose of this paper is twofold: first, it is to update those involved in the field of highway geology as to the status of registration of geologists throughout the country; second, it is to emphasize the impact that registration of geologists has on the professionals involved in highway geology. The information for the report is based on data obtained from those states that have State Boards of Registration. Each Board was contacted and the information recorded in a questionnaire which is on file at the office of the Wyoming Board for Registration of Professional Geologists.

GEOLOGIST REGISTRATION STATUS

The first state to pass a law requiring registration of geologists was Arizona in 1956. In 1968, California passed a law requiring registration of geologists with specific requirements for sub-disciplines such as engineering geology. Over the subsequent 26 years, 15 other states have enacted their own laws requiring registration of geologists. At the present time, there are 17 states with registration laws, three states advocating certification and three states defining "geologist". At least 13 other states are in the process of drafting legislation and/or submitting to the state's governing body. (See Figure 1.)

The organization known as the Association of State Boards of Geology (ASBOG) has been recently established, with presently nine states as members and several more contemplating joining. Associate members include the Association of Engineering Geologists, American Association of Petroleum Geologists and American Institute of Professional Geologists.

The purpose of ASBOG is to standardize such items as examinations, reciprocity requirements, continuing education and minimum requirements. The reason for standardization is to make it as simple as possible for the mobile geologist to practice throughout the country. This will take place over a period of time and will require combined support from all of the profession.

With the leadership of Arizona and the support of the other eight member states, an examination has been developed which could be used as a national examination. This alone has been a significant accomplishment, with four states now using the examination, and such states as California and Oregon definitely interested in utilizing the examination. Also, committees within ASBOG have been established to address other areas. These include a committee to work with the "engineer's national organization". In addition, there are committees on reciprocity and continuing education, and minimum requirements for eligibility to take the examination.

There is little doubt that registration of geologists is well established and will be required throughout the country within the near future. One reason for the emphasis on registration is the increased public awareness of their environment, and as a result, the need for engineering geologists, environmental geologists, and hydrogeologists. A good example is the registration of geologists in Wyoming, where the number of geologists involved in engineering, environment and hydrogeology are the majority compared to those practicing in petroleum and mining. (Wyoming 1994.)

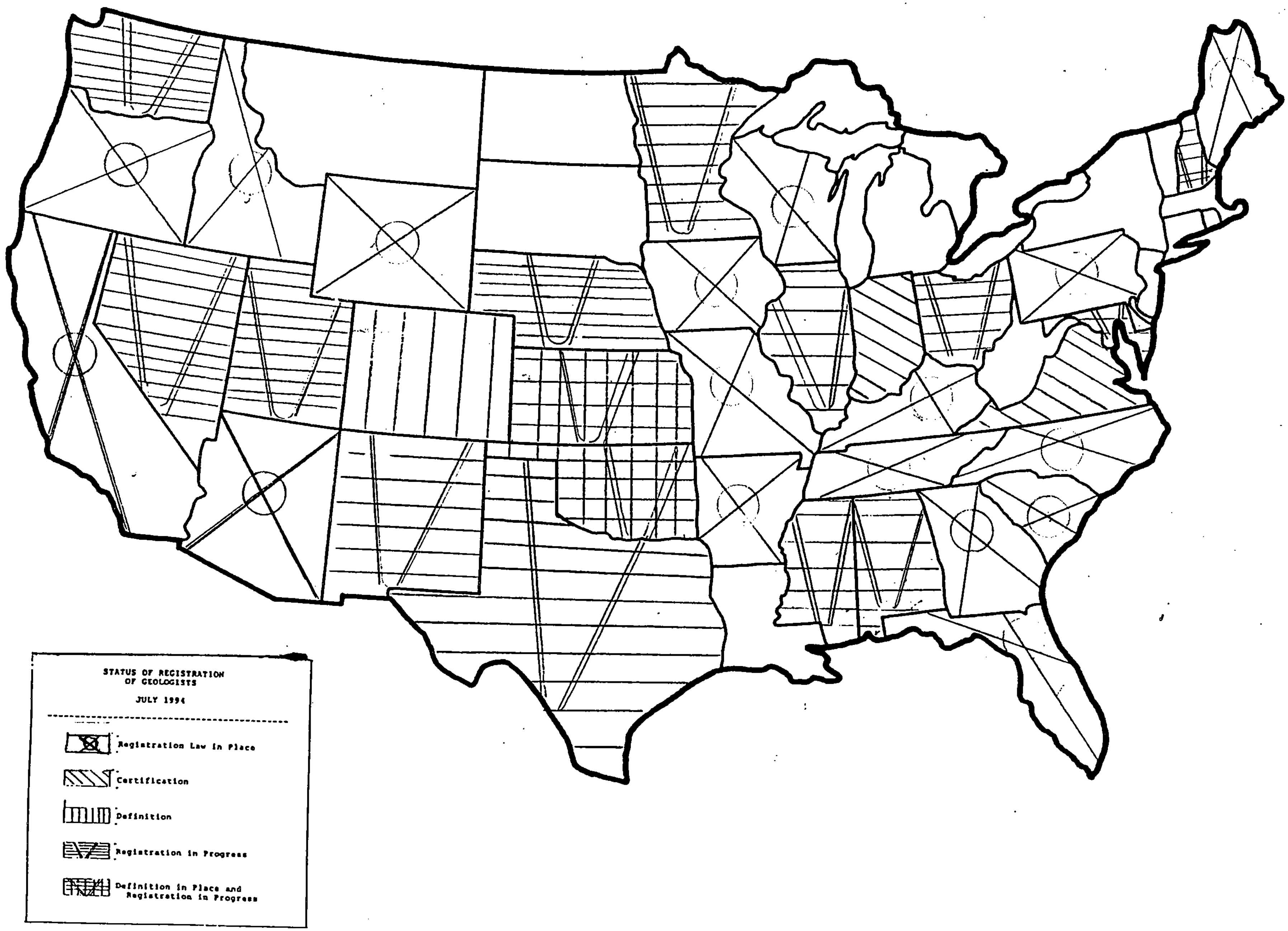
The latest figures available on registration of geologists are in the appendix. These are based on data compiled by Dr. James Williams, Acting Director and State Geologist for Missouri. (March 1994.) The information includes current registration information by state, and a list of Registration and Certification Boards of Geologists.

IMPACT OF REGISTRATION

It is important to consider the primary purpose of registration, which is the public welfare. Since the primary responsibility of the professional involved in highway geology is also the welfare of the public, the impact of registration on this field is indisputable.

Although State and Federal employees are exempt from most registration laws, becoming registered in their profession seems critical. How can there be respect and trust from the public if professionals do not make the effort to become registered? This not only improves the status

FIGURE 1.


STATUS OF REGISTRATION
OF GEOLOGISTS
JULY 1994
Registration Law in Place
Certification
Definition
Registration in Progress
Definition in Place and
Registration in Progress

of the individual in the eyes of the employer (the public), but also improves the status of the overall profession.

In the field of highway geology, geologists are required to work closely with engineers, and registration of geologists has an impact on this relationship. Registration provides the opportunity for improving the status of the geologist, but also increases their responsibility. It is then that the engineer-geologist relationship becomes even more critical.

The states vary in the way each resolves the problem of determining responsibilities between engineers and geologists. Several of the states which have separate boards of registration for geologists and engineers have a "Memorandum of Understanding" signed by both boards. Usually there is a joint committee comprised of members of both boards for the purpose of reviewing complaints involving geologists doing engineering or vise versa. This committee reports its findings and recommendations to the appropriate board. Copies of a typical "Memorandum of Understanding" may be obtained from the Wyoming Board of Registration for Professional Geologists.

In some states where one board registers both professions, there are detailed definitions of geologists and engineers which can be utilized in determining task responsibilities. A normal practice in this situation is to send the complaint to people from each profession who have been appointed as consultants. Their recommendations are submitted to the board, which makes the final ruling based on these recommendations. (Arizona 1993.)

As nearly all states and the Federal government retain consultants, a critical responsibility is to assure that these firms have registered personnel on their staff. In addition, the State and Federal personnel overseeing the consultant should be responsible to see that geologic work is done by geologists, and engineering work is done by engineers. This then requires that the State and Federal people determine task responsibility. There is little problem in determining task responsibility for tasks such as geologic mapping or design of the substructure. However, the challenge becomes more difficult in so-called "gray areas"; for example, who is responsible for the interpretation of the effect of geologic conditions on the project, who determines which strength factors to use for in-place materials, who should be signing off on the geotechnical report, etc.

The California and Wyoming boards have compiled lists of suggested task responsibilities. These lists do not have the formal approval of both geologists and engineers in either state. However, they are useful as a guide for the professional involved in the highway geology field. Copies can be obtained from the California Board of Registration for Engineers, and the Wyoming Geology Board, respectively.

CONCLUSIONS

In conclusion, it is evident that the professional people involved in the field of highway geology are as directly affected by the registration of geologists as any group. Registration does provide greater recognition for the geologist, but also increases their responsibility. It is very critical that close cooperation be maintained between the geology profession and the engineering profession, for both the welfare of the public and the good of each profession. It is in everyone's interest that decisions made regarding task responsibility be resolved by the geologist and engineer directly involved and not by some legal authority.

Since it is only a matter of time before all states will be requiring registration, or at a minimum, certification, professionals working in the highway geology field should be involved in maintaining and furthering registration in their state. Their input is especially critical in states which are right now in the process of preparing legislation for the registration of geologists.

REFERENCES

Arizona, 1993, State Board of Technical Registration Codes and Rules, Phoenix, AZ.

California State Board of Registration for Geologists and Geophysicists, 1994, Sacramento, CA, Personal Communication.

Delaware State Board of Registration, 1994, Dover, DE, Personal Communication.

North Carolina Board for Licensing Geologists, 1993, North Carolina Administrative Code, Raleigh, NC.

Oregon State Board of Geologist Examiners, 1994, Salem, OR, Personal Communication.

Williams, James H., 1994, American Association of State Geologist Professional Affairs Committee Report.

Wyoming State Board of Registration for Professional Geologists, 1994, Laramie, WY, Personal Communication.

APPENDIX

April 1994

GEOLOGIST REGISTRATION STATUS

Alabama

A registration bill was developed in 1993 with bill reported tabled in committee. Apparently no activity this year.

Alaska

Hs certification based on AIPG certification.

Arizona

Registration by nine-member board that regulates several professions. Board members include 2 architects, 3 engineers, 1 land surveyor, 1 landscape architect, 1 geologist, and 1 public member.

Arkansas

Registration has stabilized. Currently, 1747 are registered, almost 88% of registrants are out-of-state and represent almost every other state and several foreign countries. Board consists of 5 geologists, the State Geologist, and 1 public member. Currently the GRE is used for examination qualifications. Scheduling is arranged by the applicant with the university.

California

Only state registering geologists and geophysicists as separate professions. Board has 2 geologists, 1 geophysicist, and 5 public members. The Board certifies licensed geologists as Certified Engineering Geologists and will offer a hydrogeologist certification in Spring 1995. All licenses and certificates are by examination.

Colorado

No changes; has definition.

Connecticut

No registration, certification, or definition and no change reported.

Delaware

State Boards expect some changes due to strict limitations or term length. On the geologist board, there are three professionals (State Geologist is ex officio), and two public members. A new public member just appointed and a professional to be replaced soon. Delaware General Assembly now requiring all professional regulatory boards to become self supporting causing substantial fee increases. Certification limitations relative to states with examination licensure hinders reciprocity. Statute change possibilities to examination requirement are nil.

Florida

Board of Professional Geologists have 8 members: 5 geologists, 2 lay persons, and the state geologist (ex officio). Currently, 1,673 registrants and 187 certified geology firms. Recognizes endorsement with the following states: AZ, CA, GA, ID, MA, OR, SC, VA, and WY, provided applicants meet minimum FL requirements and have taken and passed at least one exam in one of the named states. With exam acceptance, KY and PA to be added when they offer exam. Applicant also must take FL rules portion of the exam.

Georgia

Georgia has signed a reciprocity agreements with South Carolina, North Carolina and Virginia. Board consists of 5 geologists, 1 public member, and the State Geologist.

Hawaii

No changes reported.

Idaho

Has registration with no changes reported.

Illinois

A consensus bill, developed through discussions between Illinois chapters of geological professional societies and several agencies of state government, has twice been passed by the General Assembly and vetoed by the governor. Through some further discussions, the bill has been slightly modified in an attempt to overcome some of the objections raised in the governor's veto messages. The Professional Geologist Licensing Act, House Bill 3536, was introduced in the General Assembly on March 17, 1994.

Indiana

The Certification of Professional Geologists Program has been transferred from the Indiana Department of Natural Resources to the Indiana Geological Survey at Indiana University. In collaboration with "The Professional Geologists of Indiana, Inc." planning has been initiated for the development of a Board and the added requirement of examination. Present plans are for a bill to be written and introduced during the 1995 state legislative session.

Iowa

Registration required by Iowa DNR for certain groundwater professionals who provide services at leaking underground storage tanks.

Kansas

Registration legislation hearing, November 1993, but mixed reviews in committee. Now supported by petroleum geologists, however, retraining for work in other aspects of geology, especially environmental geology, must be considered. Plan to reintroduce in January 1995.

Kentucky

The Kentucky registration law was developed using the AIPG-AEG model guide. Board consists of four geologists, one of whom is the State Geologist and one public member. As of April 1994, there were 1,805 geologists registered. Kentucky plans on using the NASBOG national exam.

Louisiana

No registration activity.

Maine

Registrant testing about four times per year. Two part examination, 6-8 hour length, including regional knowledge, environmental geology, and a number of specialties. Specialty examples include hydrology, marine, sedimentation, and structure. Applicants take two of the specialties in the examination process. No specialty registration. More than 300 registrants. Joint geology and soil scientist registration board with separate qualifications for each. Board consists of 2 geologists, 2 soil scientists, State Geologist, and 1 public member. The State Geologist and the state salaried soil scientist are ex officio.

Maryland

No registration, certification, or definition. The Capitol AIPG section has proposed a registration bill in the beginning stages.

Massachusetts

No registration, certification, or definition. A land council statute reportedly includes a reference to geologists.

Michigan

No registration activity. Geologist defined broadly in state hazardous waste regs (Act 64) and definitions are proposed for new rules relative to solid waste (Act 641) and oil and gas regulations (Act 61). Engineers are commonly calling themselves engineering geologists or geological engineers doing groundwater clean-up work without knowledge of geologic processes.

Minnesota

No registration, certification, or definition. A registration bill has been proposed, but has not yet been introduced to the State Legislature. The proposed legislation would include geology into an existing board of registration. The proposed board would consist of 3 architects, 5 engineers, 1 landscape architect, 2 land surveyors, 6 public members, and 1 geologist.

Mississippi

No registration, certification, or definition requirements exist.

Missouri

Currently definition only. Draft bill using CoPGO model with AEG-AIPG and other state modifications, SB 649, has been introduced and voted "Do Pass" out of Senate Committee. Bill pertains to geologic work relative to health, safety, and welfare. The Senate sponsor, Senator Jerry Howard, indicates present bill assignment of 16th on Senate agenda might allow adequate time. The House sponsor, Rep. Jerry McBride, is awaiting bill from the Senate. Passage is a slim possibility this session.

Montana

Limited discussions continue about registration. No interest or other activity at present.

Nebraska

Nebraska Board of Examiners for engineers and architects updating statutes. Concurrently, the Nebraska Geological Society, having excellent rapport with the engineering community and the Board, may finalize legislation this year by adding geologist registration. The Board would then be known as Board of Examiners for Engineers, Architects, and Geologists.

Nevada

AEG section and other geological organizations preparing registration legislation possibly limited to engineering geologists. Excludes exploration and mining geologists. May be added to Nevada Engineering and Land Surveyor Board of Registration.

New Hampshire

Legislation to register geologists abandoned in 1993. There is little probability will be reintroduced in foreseeable future.

New Jersey

No changes and no activity foreseen in near future. There is a certification program based on UST regulations that includes definition of qualified groundwater consultants.

New Mexico

Engineering and environmental geologists advocating registration; most petroleum and university people oppose registration.

New York

No registration, certification, or definition requirements exist. Initial registration efforts beginning with professional organizations of geologists in Buffalo and Syracuse.

North Carolina

Board consists of 4 geologists, State Geologist (ex officio), and 1 public member. Board pursing comity agreements, presently have comity with SC, GA, and VA. A 1991 amendment brought geologists under Professional Corporation Act; board licensing P.C.'s offering geological services. Active NASBOG member. NC gives examination twice a year.

North Dakota

John Bluemle has drafted a bill using the Wyoming statute and the Missouri legislation proposal as information sources. If these appear to be supported, he will provide the draft to the local geological society for the 1994-95 legislative session.

Ohio

Proposed registration bill. Uses applicable portions of AIPG-AEG model law version. Oil geologists opposed but bill emphasized environmental protection.

Oklahoma

A geologist/geology definition act, HB 1167, was enacted and signed into law in 1993. AIPG is assisting in work to amend the definition law into a registration act.

Oregon

Oregon is one of two states with engineering geology specialty registration. Joint staff support exists between the registration board for geologists and for engineers; fosters good professional working relationships. Board consists of 4 geologists, the State Geologist (ex officio), and 1 public member. The Geological Department of Portland State University offers a review course for examination preparation.

Pennsylvania

Registration Act 151 was passed by the Pennsylvania legislature December 21, 1992. Joined with the Engineering Board of Registration as an amendment forming a combined geologist-engineer board. Board consists of 2 geologists, 5 engineers, 2 land surveyors, 3 public members, and the Commission of Professional Occupation Affairs. Even though combined with engineering registration, geologists position retains basic concepts of the model law. Governor Casey signed bill on 3/31/93. The grandfather clause lasts for two years. Oil and gas exploration exempted. Contact State Board of Registration for further information.

Rhode Island

No registration, certification, or definition requirements exist and no changes reported.

South Carolina

South Carolina and Georgia signed a reciprocity agreement on April 21, 1992. The State Board of Registration has continuing education guidelines. Like Oregon, South Carolina is developing liaison between board of geologists and engineering registration. An MOU has been developed.