46 TH
HIGHWAY GEOLOGY
SYMPOSIUM

CHARLESTON, WEST VIRGINIA
MAY 14-17, 1995

PROCEEDINGS

Sponsored By

West Virginia Geological &
Economic Survey
West Virginia Division of Highways
West Virginia University,
Dept. of Geology
Transportation Research Board

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 45 consecutive annual meetings have been held in 31 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
1993	Tampa, FL	1994	Portland, OR

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

A number of three-member standing committees conduct the affairs of the organization. The lack of rigid requirements, routing, and the 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 in the greater Louisville area. The 1986 field trip was through the Rockies on 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.

HGS

STEERING COMMITTEE OFFICERS

Mr. Harry Moore, Chairman Geologic Eng. Supervisor I Division of Materials and Tests Bureau of Operations Department of Transportation P. O. Box 58 Knoxville, Tennessee 37901 PH: 615/594-6219	1995
Mr. Earl Wright, Vice Chairman Engineering Geology Section Supervisor Geotechnical Branch Division of Materials Department of Highways Transportation Cabinet Frankfort, Kentucky 40622 PH: 502/564-2374	1995
Dr. C. William Lovell, Secretary School of Civil Engineering Purdue University West Lafayette, Indiana 47907 PH: 317/494-5034	1995
Mr. Russell Glass, Treasurer Area Geologist Geotechnical Unit N.C. Department of Transportation P. O. Box 3279 Asheville, North Carolina 28802 PH: 704/298-3874	Appointed by Chairman

NOTE: Officers' terms expire at conclusion of 1995 Symposium.

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* deceased

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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" medallian mounted on a walnut shield and appropriately inscribed, is presented during the banquet at the Annual Symposium.

* deceased

46th HIGHWAY GEOLOGY SYMPOSIUM

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TRANSPORTATION RESEARCH BOARD SYMPOSIUM

Managing Rock Fall Hazards: Identification, Prioritization, and Mitigation.

SUNDAY MAY 14, 1995

7:30 a.m. Registration, Coffee, Kanawha Foyer

8:20 a.m. Welcome and Introductions
G. P. Jayaprakash, Transportation Research Board; Jeffrey R. Keaton, AGRA Earth & Environmental; A. Keith Turner, Colorado School of Mines

Technical Session - Session Moderator:
Michael P. Vierling, NYDOT

8:30 a.m. **Overview of the Federal Rock Fall Management Program.** Barry Siels, Federal Highway Administration

8:55 a.m. Identifying Road Fall Hazards: an Example from Glenwood Canyon, Colorado. Richard Andrew, Colorado Department of Highways

9:20 a.m. Characterizing Rock Fall Hazards. Skip Watts, Radford University

9:45 a.m. **Prioritizing Hazardous Rock Slopes: The Oregon DOT Experience.** Larry Pierson, Oregon Department of Transportation

10:10 a.m. **The New York State Approach to Prioritizing Rock Fall Remediation.** Richard Grana & Michael P. Vierling, New York State Department of Transportation

10:30 a.m. **Coffee Break**

10:55 a.m. **Cost-Effective Rock Slope Stabilization Techniques.** Duncan C. Wyllie, Golder Associates

11:20 a.m. **Catching Rocks Before They Hit the Road-The Ritchie Ditch Revisited.** Larry Pierson, Oregon Department of Transportation

11:45 a.m. **Stopping Rocks on the Slope - A Review of Fences and Barriers.** John D. Duffy, California Department of Transportation

12:10 p.m. **Buffet Lunch**

1:30 p.m. **Rock Fall Simulation: an example from Glenwood Canyon, Colorado.** Jerry D. Higgins, Colorado School of Mines

1:55 p.m. **Modeling Fences and Barriers for Optimum Design.** George Hearn, University of Colorado

2:20 p.m. **Field Trip Preview and Logistics.** Glen Sherman, West Virginia Division of Highways

2:45 p.m. **Break**

3:00 p.m. **Field Trip Departs**

Stop 1: Fort Hill Slope, I-64 Offramp, Charleston, WV. Glen Sherman, WVDOH

Stop 2: Problem Slope, Junction of I-77 and I-79, Charleston, WV. Glen Sherman, WVDOH

6:00 p.m. Arrive back at Marriott, End of TRB Symposium

5:00 p.m. - 9:00 p.m. **REGISTRATION - Ballroom Foyer**

7:00 p.m. - 9:00 p.m. **ICEBREAKER RECEPTION-**
Exhibit hall, Salons A/B

MONDAY MAY 15, 1995

8:00 a.m. - 5:00 p.m. **REGISTRATION**- Ballroom Foyer

8:00 a.m. - 5:00 p.m. **GUEST PROGRAM** - meet in Ballroom Foyer for coffee & danish

8:00 a.m. **COFFEE & DANISH,** Ballroom Foyer

8:00 a.m. - 6:00 p.m. **EXHIBITOR DISPLAYS** - Salons E&F

8:30 a.m. **TECHNICAL SESSION,** Salon C
Session Moderator: John Baldwin

8:30 a.m. **Welcoming remarks**:
Ken Ashton, West Virginia Geological Survey
John Baldwin, West Virginia Div. of Highways
HGS Steering Committee

9:00 a.m. **The Geology of West Virginia.** Dr. Bob Behling, Dept. of Geology, West Virginia University

9:40 a.m. **A Geotechnical Investigation of the Chagrin River Road Landslide Complex in the Moreland Hills Area, Cuyahoga County, Ohio.** Mark A. Kroenke and A' Shakoor, Kent State University

10:05 a.m. Coffee Break

10:35 a.m. **Potential for Slope Instability and Acidic Runoff along the U.S. 19 Corridor; Cherokee, Graham, and Swain Counties, North Carolina.** Benjamin C. Reed, W. R. McConnell, and D. M. Mullen, North Carolina Department of Transportation

11:00 p.m. **Mitigation Landslide/Rockfall Hazards- A Quantitative Evaluation of Risk Reduction.** Christopher A. Ruppen and Gordon M. Eliot, Michael Baker Jr., Inc.

11:25 p.m. **Landsliding in Pennsylvania.** James V. Hamel and Elizabeth A. Hamel, G Tech Inc.

11:50 p.m. **LUNCH,** Executive Buffet, Salon D

TECHNICAL SESSION II, Salon C
Session Moderator: J. Steven Kite

1:20 p.m. **Maryland Route 31 Sinkhole.** A. David Martin, Maryland Dept. of Transportation/State Highway Administration

1:45 p.m. **Remediation for Highways in Karst.** Joseph A. Fischer and Joseph J. Fischer, Geoscience Service

2:10 p.m. **Management of the Discharge Quality of Highway Stormwater Runoff in Karst Areas to Control Impacts to Ground Water- A Progress Report**. J. Brad Stephenson, Dr. Barry F. Beck, Dr. James L. Smoot, and Anne Turpin, P. E. LaMoreaux & Associates, Inc.

2:35 p.m. **Landslide Repair Utilizing A Steepened Geogrid Reinforced Slope, Interstate 90, M.P. 28.9, Sheridan County, Wyoming**. G. Michael Hager and Mark Falk. Wyoming Department of Transportation

3:00 p.m. **Coffee Break**, Exhibit Hall

3:30 p.m. **Design of Rock Reinforcement System for Tallulah Falls Bridge Foundations.** Yash P. Singh, Bruce P. Cavan, and Gary W. Rhodes, Law Environmental, Inc.

4:55 p.m. **Rockfall Mitigation Using Wire Nets**. Joseph C. Bigger, Brugg Cable Products, Inc.

4:20 p.m. **Helical Tieback Anchors Help Reconstruct Failed Sheet Pile Walls.** Gary L. Seider and Walter Smith, A. B. Chance Co.

4:45 p.m. **Permanent Highway and Landslide Walls**. Harald P. Ludwig, Schnabel Foundation Company

6:30 p.m. - 9:30 **OPTIONAL DINNER CRUISE,** P. A. Denny Sternwheeler, Depart from Marriott Lobby

:SDAY MAY 16, 1995

FIELD TRIP

8:00 Depart from Marriott Lobby

9:00 Stop 1 - Memorial Tunnel, West Virginia Turnpike, Fire and ventilation test project

Stop 2 - Cut and Fill, West Virginia Turnpike

12:00 - 1:30 LUNCH - Hawks Nest State Park
Lunch Program - The New River Gorge, National Park Service.

2:00 Stop 3 - New River Gorge, Base of bridge

3:00 Stop 4 - New River Gorge National River Visitors Center, top of gorge

5:00 Return to Marriott

5:30 p.m. **H.G.S. STEERING COMMITTEE MEETING** - Allegheny Room

6:30 p.m. **BANQUET SOCIAL HOUR**, Salons A/B

7:30 p.m. **BANQUET**, Salons D/E/F

8:30 p.m. Banquet Program - **The History of Transportation in West Virginia,** Fred Armstrong, West Virginia Division of Culture and History.

WEDNESDAY, MAY 17, 1995

TECHNICAL SESSION III - Salon C
Session Moderator: Jim Fisher

9:00 a.m. **Limestone Base Course Permeability.** Chin Leong Toh and Sam Thornton, University of Arkansas

9:25 a.m. **Uses of Scrap Rubber Tires in Civil Engineering**. C. W. Lovell, Andres Bernal, and Rodrigo Salgadado, Perdue University

9:50 a.m. **Evolution of A Technique: Petrography of Aggregates for Concrete and Bituminous Highway Pavements.** Terry R. West, Perdue University

10:15 a.m. Coffee Break, Exhibit Hall, Salons A/B

10:45 a.m. **A Modified Presplit Blasting Method for Use in Environmentally Sensitive Areas.** R. Michael Johnson and Michael P. Vierling, New York State Dept. of Transportation

11:10 a.m. **Effects of Soil Horizonation on Flexible Pavement Responses.** Kevin D. Hall and Quintin B. Watkins, University of Arkansas

11:35 a.m. **Prediction of the Unconfined Compressive Strength of Carbonate Rocks from Los Angeles Abrasion Test Data, Christopher L. Brown, Gannett Fleming, Inc, and Abdul Shakoor, Kent State University**

12:00 a.m. Official Close of 46th Highway Geology Symposium

THURSDAY, MAY 18, 1995

8:00 a.m. **OPTIONAL FIELD TRIP -**
Geology of the New River Gorge, Raft trip
Depart from Marriott Town Center Lobby

6:00 p.m. Return to Marriott Town Center, Charleston, West Virginia

46TH HIGHWAY GEOLOGY SYMPOSIUM

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A GEOTECHNICAL INVESTIGATION OF THE CHAGRIN RIVER ROAD LANDSLIDE COMPLEX IN THE MORELAND HILLS AREA, CUYAHOGA COUNTY, OHIO

Mark A. Kroenke and Abdul Shakoor
Department of Geology and The Water Resources Research Institute,
Kent State University, Kent OH 44242

ABSTRACT

Northeast Ohio is an area of moderate to high landslide susceptibility and incidence. Nowhere is this more evident than along the Chagrin River Valley where steep valley walls are composed of weak glacio-lacustrine silts and clays. The Chagrin River Road landslide complex was selected for detailed investigations because it is representative of slope failures impacting roadways and homes along the Chagrin River Valley. The term complex is used because the site involves more than one landslide. The most prominent of these landslides was mapped using a transit to depict landslide features and topography. Slope movement was monitored for a period of one year. A detailed soil profile for the site was established using a combination of visual observations and borehole data, and undisturbed block samples were collected for lab tests. Soil samples were tested to determine grain-size distributions, Atterberg limits, and strength parameters. These properties were used to characterize the soils from this site and to analyze the stability of the landslide complex using the STABL program.

Results of the study indicate the Chagrin River Road slope failure is a combination of slump and flow movements occurring in the glacio-lacustrine silty clays (CL) which have peak friction angles ranging from 19.5 to 21 degrees and peak cohesion values from 108 to 126 psf. Slope monitoring yielded movements up to 3 feet signaling continued instability of the slope. The stability analyses support the conclusion that water is a significant factor in initiating and perpetuating the failure. Human activity has accentuated the slope movement through oversteepening of the slope and cutting of the toe during the construction of Chagrin River Road, and overloading the top of the slope with a housing development. Chagrin River Road has been closed several times since 1991 due to the active nature of this slide complex.

INTRODUCTION

Northeast Ohio has been classified as a region of high landslide incidence and susceptibility (Radbruch-Hall et al., 1982). Nowhere in northeast Ohio is this high incidence of landslides more apparent than in Cuyahoga County (Figure 1). Most slope movements in Cuyahoga County are concentrated along the gorge-like valleys of the Cuyahoga and Chagrin rivers (White, 1982). While slope movements along the Cuyahoga River Valley have received some attention and study (Gardner, 1972; Jones and Shakoor, 1989), slope movements in the Chagrin River Valley have received little or no geologic scrutiny.

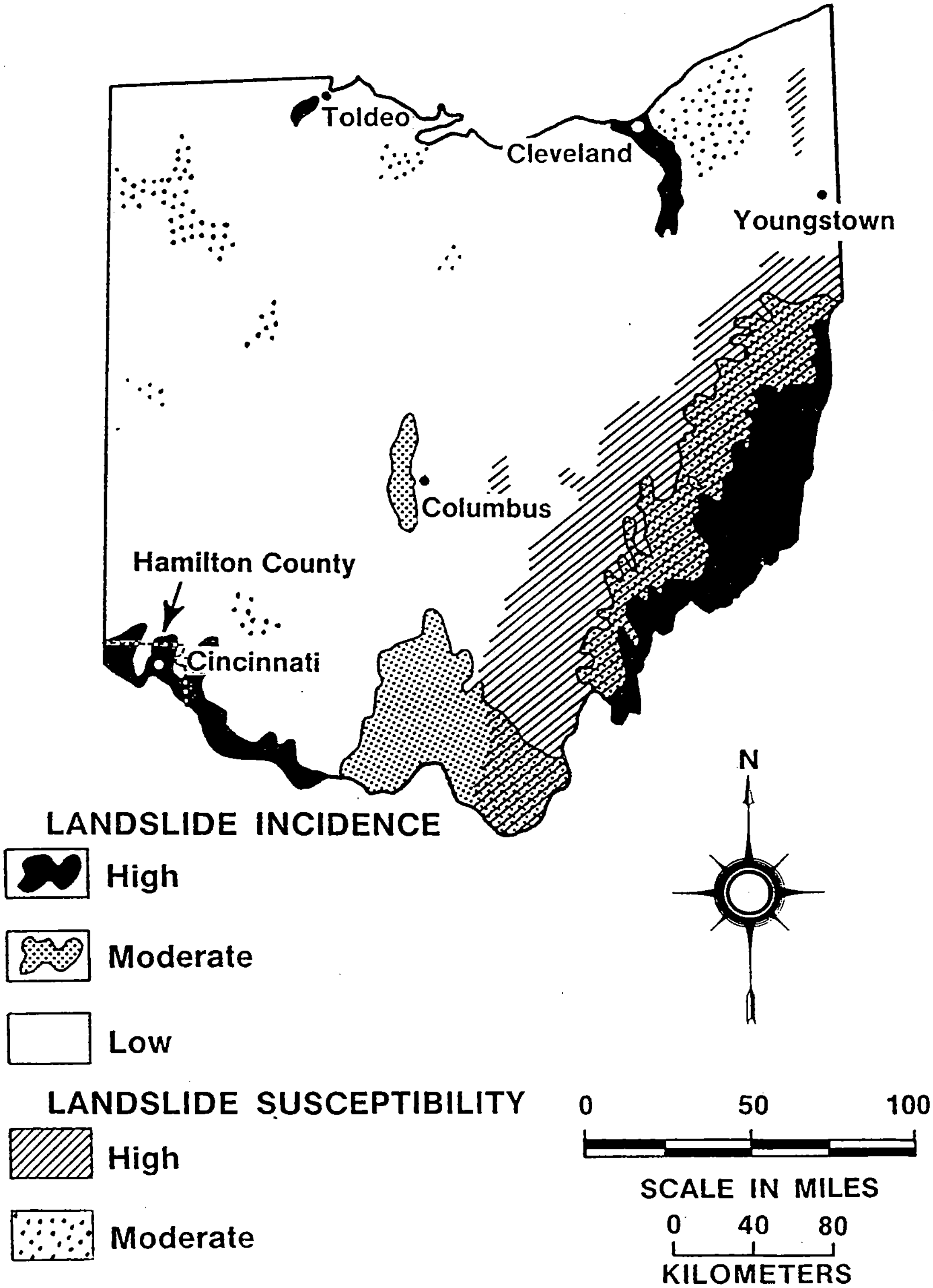


Figure 1: Location of Cuyahoga County in relation to landslide-susceptible areas within the State of Ohio (modified from Radbruch-Hall et al., 1982)

The Chagrin River Valley has a high incidence and susceptibility to slope movements for a number of reasons. First, advance and retreat of ice sheets throughout the Pleistocene Epoch led to natural damming of the Chagrin River (Ford, 1987). This temporary damming produced a lacustrine environment and subsequent deposition of abundant silt and clay (Ford, 1987). Secondly, increased rates of erosion and downcutting that followed deglaciation produced steep valley slopes. Finally, northeast Ohio's humid climate provides plenty of water through rain and snowmelt to cause piping in the silty soils and loss of strength in the clayey soils.

The combination of steep slopes, abundant glacio-lacustrine silts and clays, humid climate, and alteration of the landscape through human activity has resulted in a number of damaging slope movements in the Chagrin River Valley. The objective of this study was to perform a detailed geotechnical investigation of the Chagrin River Road landslide complex and determine its impact on travel and residential development in the region.

METHODOLOGY

Field Investigations

The Chagrin River Road site was mapped with a transit to accurately depict physiographic features such as scarps, tension cracks, groundwater seeps, and ponded water. In addition to mapping, the slope movement at the Chagrin River Road site was monitored from July, 1993 to June, 1994. Three transects of wooden stakes were placed along the long axis of the slide, and designated as the western, middle, and eastern transects (Figure 2). The bases of three stable (vertical) trees, 30-50 feet (9-15 m) from the upper scarp face, were designated as the reference points for the three transects. The distance the stakes moved with respect to the stable reference point was measured on a bimonthly schedule throughout the year.

Chunk samples of cohesive soils and bulk samples of granular soils were collected for lab testing. This was accomplished by first establishing the soil stratigraphy of the site, and then taking representative samples from each stratum for laboratory testing. In addition to field observations and lab tests, drilling information from previous investigations was used to complete the characterization of subsurface geology on site.

Laboratory Testing

Grain size distributions and Atterberg limits were determined for all samples in order to classify them according to the Unified Soil Classification System. The direct shear tests and blow count (n) values were used to determine the strength parameters for stability analysis. All tests were performed in accordance with the American Society for Testing and Materials procedures (ASTM, 1993).

Figure 2: Photograph showing the middle transect of stakes with the middle and upper regions of the slide mass as the backdrop.

Stability Analysis

The stability analyses were conducted using the STABL4M program. The program assumes a circular surface of failure and calculates the factor of safety by the Simplified Bishop Method of Slices. STABL requires information regarding slope geometry, position of the water table, soil stratigraphy, and values of density, cohesion, and friction for individual soil layers. The original slope was analyzed for both the dry and saturated conditions, and factors of safety were assigned respectively. The existing slope was then similarly analyzed to determine to what extent failure is still possible under varying drainage conditions.

DESCRIPTION AND ANALYSIS

The Chagrin River Road Landslide Complex is located within the Chagrin River Valley in Cuyahoga County, Ohio (Figure 3). The landslide complex is a hazard to Chagrin River Road, which has been closed several times since 1991 due to slide activity. The most prominent of these landslides was mapped to depict slide dimensions, scarps, tension cracks, earth flows, ponded water, and seepage (Figure 4).

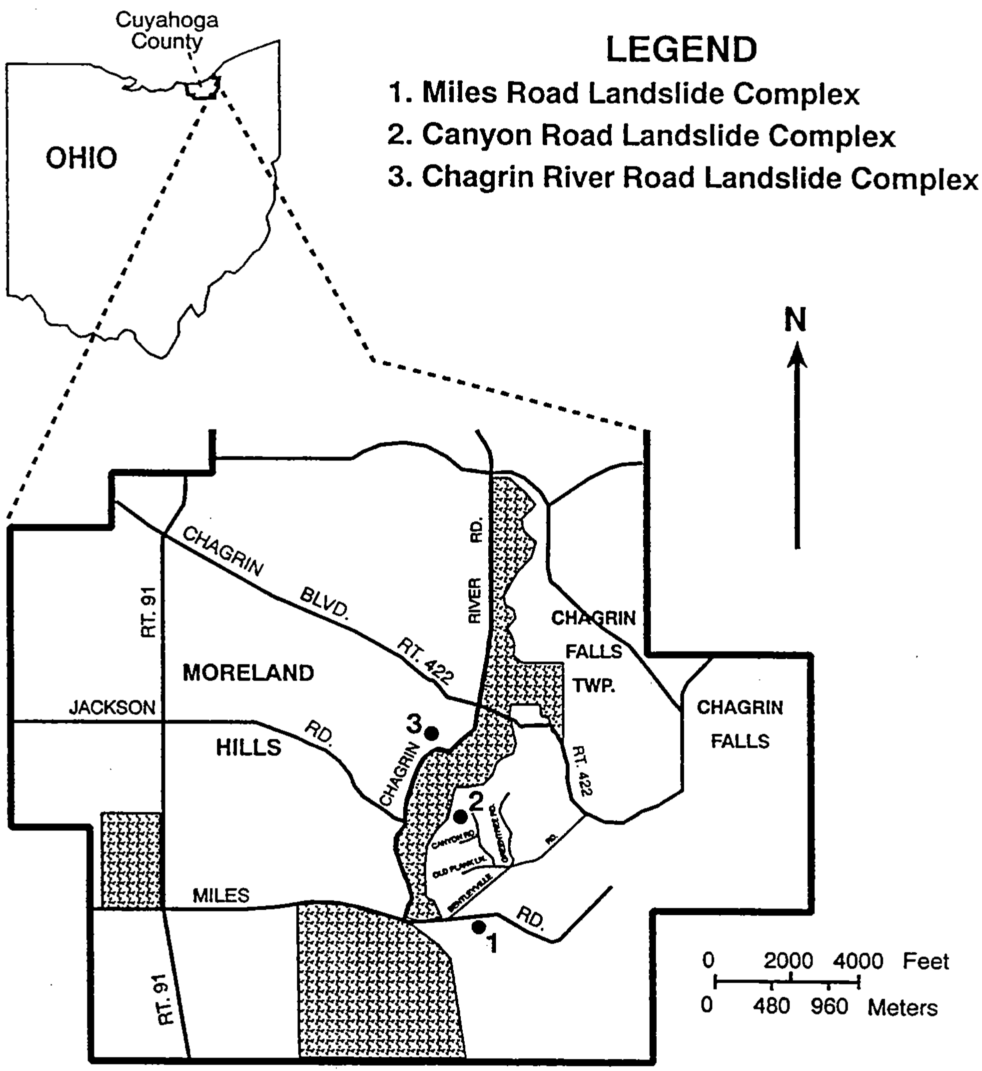


Figure 3: Location of Chagrin River Road Landslide Complex and other landslide complexes in Moreland Hills.

The site stratigraphy consists of 60 feet (18.3 m) of silty clay (CL) overlying sand (SP) and gravel (GW) (Figure 5). The slope failure is a combination of slump and flow movements (Varnes, 1978), primarily occurring within the silty clay. A small spring emerges from the base of the sand layer and forms a small stream that flows down over the toe of the slide and into a sediment choked drainage culvert (Figure 6). Poor maintenance of this drainage culvert prevents proper drainage of the toe of the landslide and contributes to slope instability.

Monitoring of the slide from July 1993 to June 1994 has shown that the site is still unstable. Movements of up to 3 feet (0.9 m) were recorded along the eastern transect, while movements of 2 to 2.5 feet (0.6-0.75 m) were observed along the west and middle transects respectively. Plots of movement versus time for the eastern transect are shown in Figure 7.

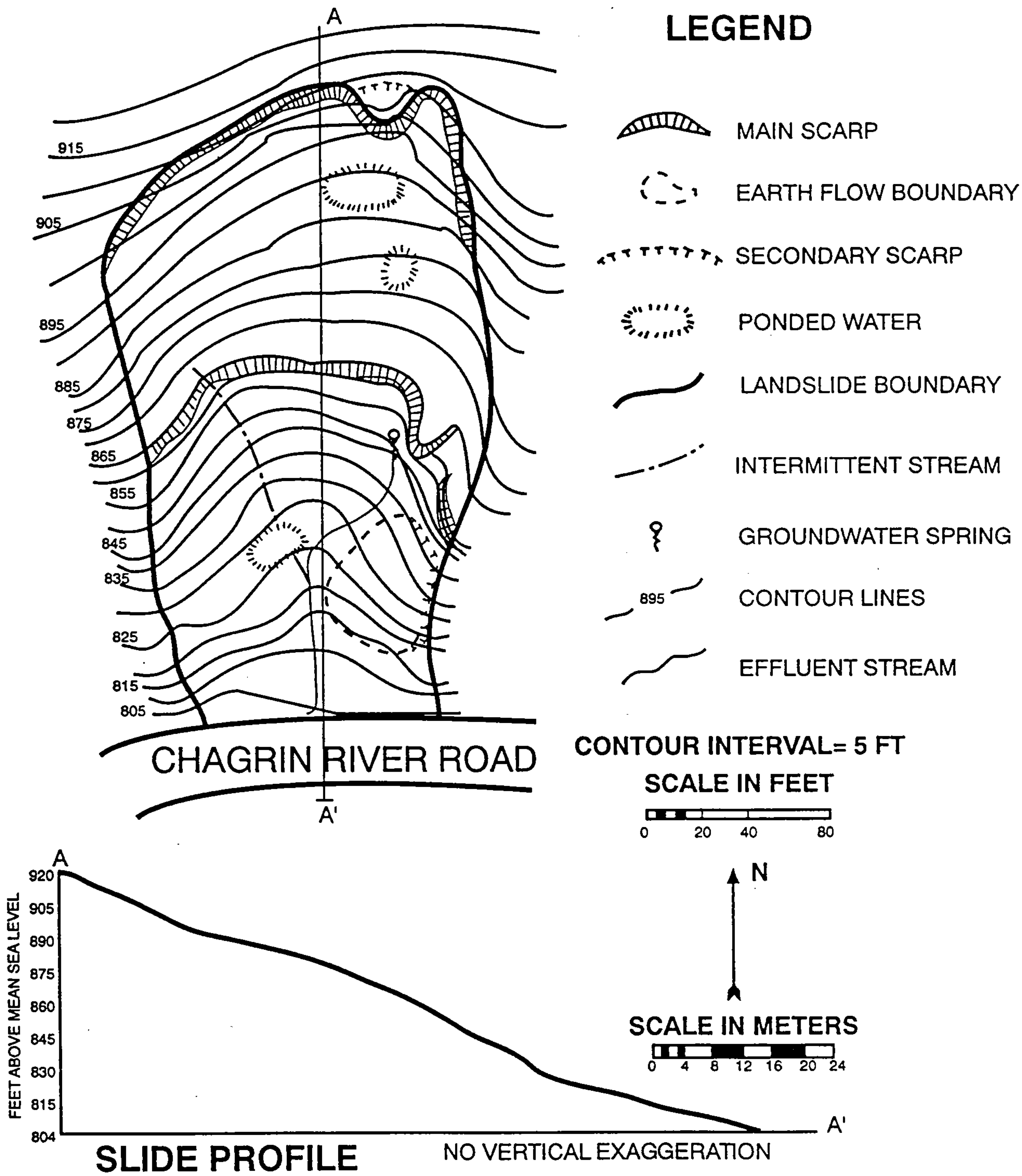


Figure 4: Topographic map of the most recently active landslide of the complex, depicting physiographic features.

Depth (ft)	Profile	USCS	Soil Description	Engineering Properties						
				Wn	LL	PI	N_{avg}	ø	C (Psf)	γ_m (Pcf)
0-10		CL	Brown silty clay	28	43	20	15	21.5°	126	123
10-60		CL	Gray silty clay	30	42	19	21	19.5°	108	120
60-70		SP	Brown sand	12			30	30°*		120*
70-105		GW	Coarse gravel and sand	12			55	35°*		130*

* Determined from n values

Figure 5: Stratigraphy and engineering properties of the various soil layers.

Figure 6: Sediment choked drainage ditch, which has forced water to drain over Chagrin River Road.

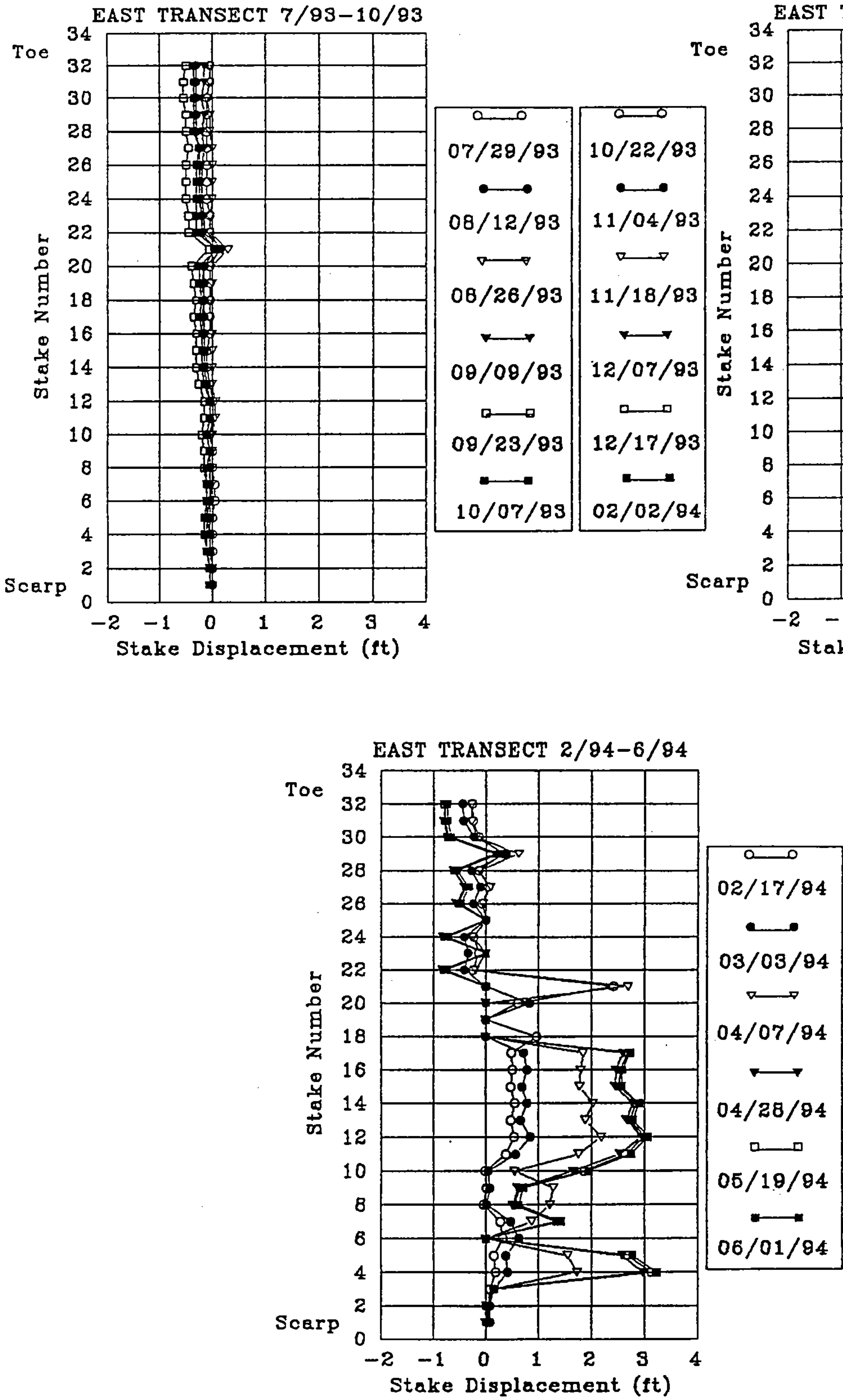


Figure 7: Plots of movement versus time for the eastern transect, from 7/93 to 6/94.

Plots of the western and middle transects reveal similar trends. The plots show that most of the movement is occurring in the middle to upper regions of the slide mass, where steep slope angles in weak soils are prevalent.

Figure 8 shows the results of the stability analyses for the original slope at the site. The factor of safety for the dry condition is 1.2, while the factor of safety for the saturated condition is equal to 1.0. The results of the stability analyses show the significance of water in initiating and perpetuating the slope failures at this site. Figure 9 shows the results of the stability analyses for the existing slope at the site. The factor of safety for the dry condition is 1.17, while the factor of safety for the saturated condition is equal to 1.0. These results signal that the slope is not yet at equilibrium. In fact, failure of the original slope has left the remaining slope even more unstable with an increased hazard to the Chagrin River Road.

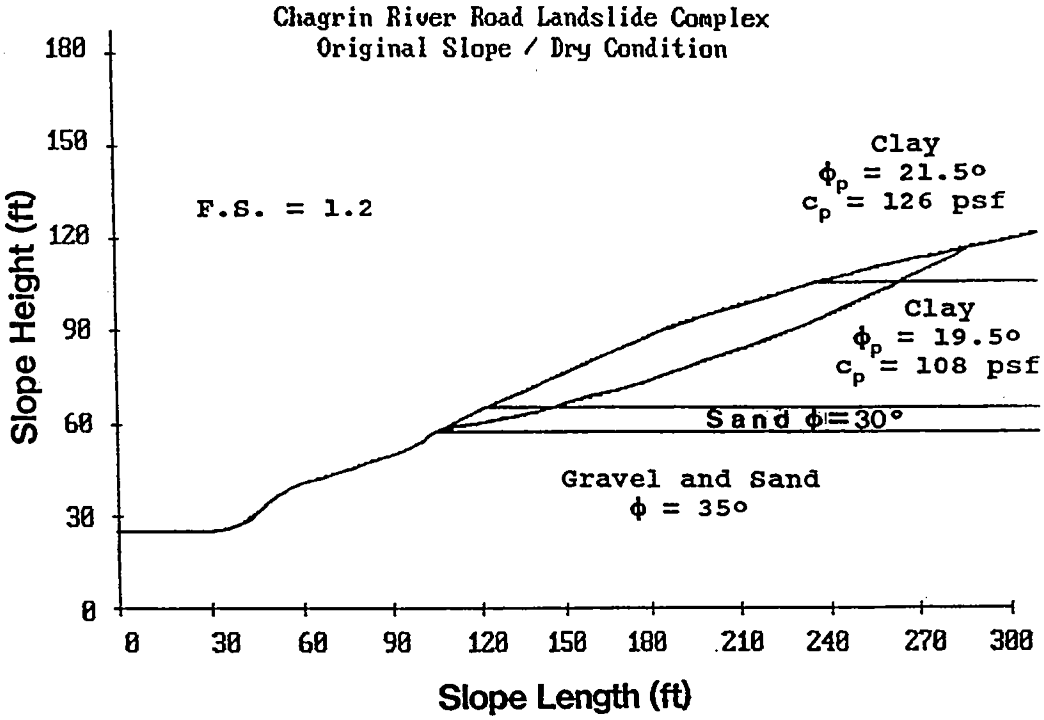
Previous remedial measures taken to stabilize the slope included the removal of slide material at the toe and the installation of subdrains within the sand and gravel layers (Davis, 1993). Upon installation of the subdrains water was released from the slope at a rate similar to a gushing fire hydrant for approximately 24 hours, after which the flow rate gradually decreased (Davis, 1993). However, the presence of the small spring currently emerging from the base of the sand layer suggests the ability of the subdrains to effectively relieve groundwater pressure in the slope has been compromised by some sort of blockage. Removal of slide material at the toe, albeit the least expensive corrective measure, has removed lateral support for the slope and has facilitated the threat of future slope failures.

ENVIRONMENTAL IMPACTS AND REMEDIAL MEASURES

The use of minimum corrective measures has nevertheless prevented large potentially hazardous slope movements thus far. Unfortunately, the site still poses a slope stability problem as the results from the stability analyses and the stake monitoring show. A prolonged precipitation event could provide enough moisture to raise the water table above the critical level and cause failure.

The landslide complex has affected not only travel on Chagrin River Road, but a half million dollar residence at the crest of the slope has been refused an occupancy permit by the Village of Moreland Hills (Davis, 1993). The homeowner has had many difficulties obtaining homeowners insurance as a result (Davis, 1993). The development of Mountain Run Estates, in the vicinity of the slide complex, has been impeded because the village engineer for Moreland Hills has denied the developer permission to build on lots adjacent to the current hilltop residence, citing concerns that the slope will be further destabilized resulting in failure (Hippley, 1993).

The site is suitable for the construction of a soldier beam type retaining wall near the toe area, with a wedge shaped gravel filter for drainage purposes. The filter should be wrapped in a geofabric to prevent piping of the fine grained soil material. This type of retaining structure has successfully stabilized a landslide of similar dimension in the Pittsburgh area (Figure 10).



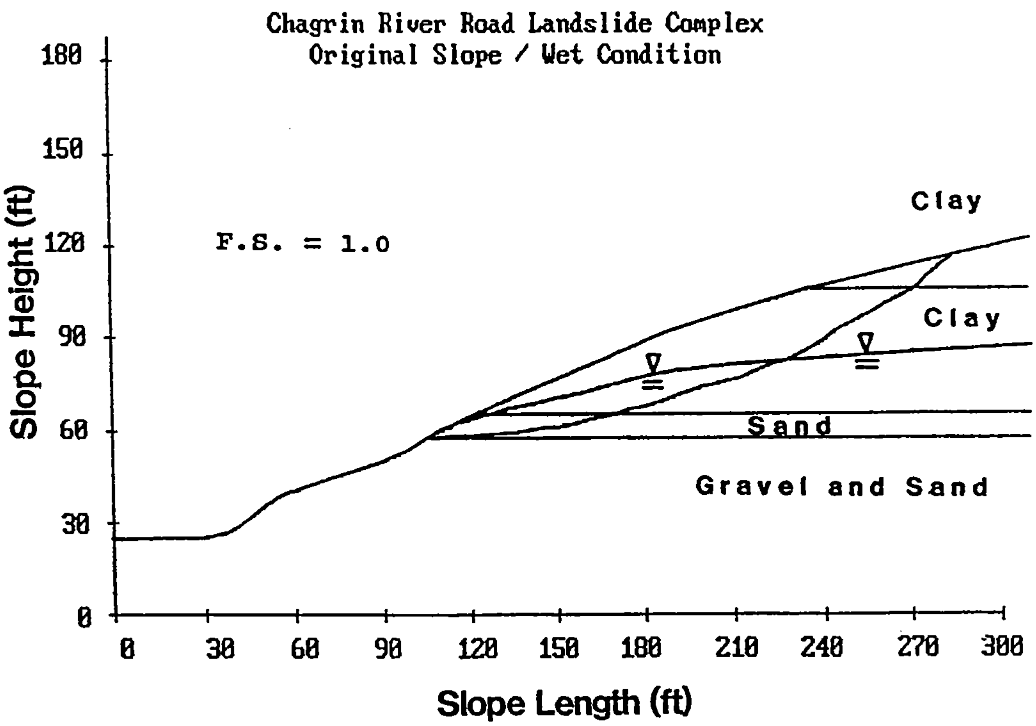
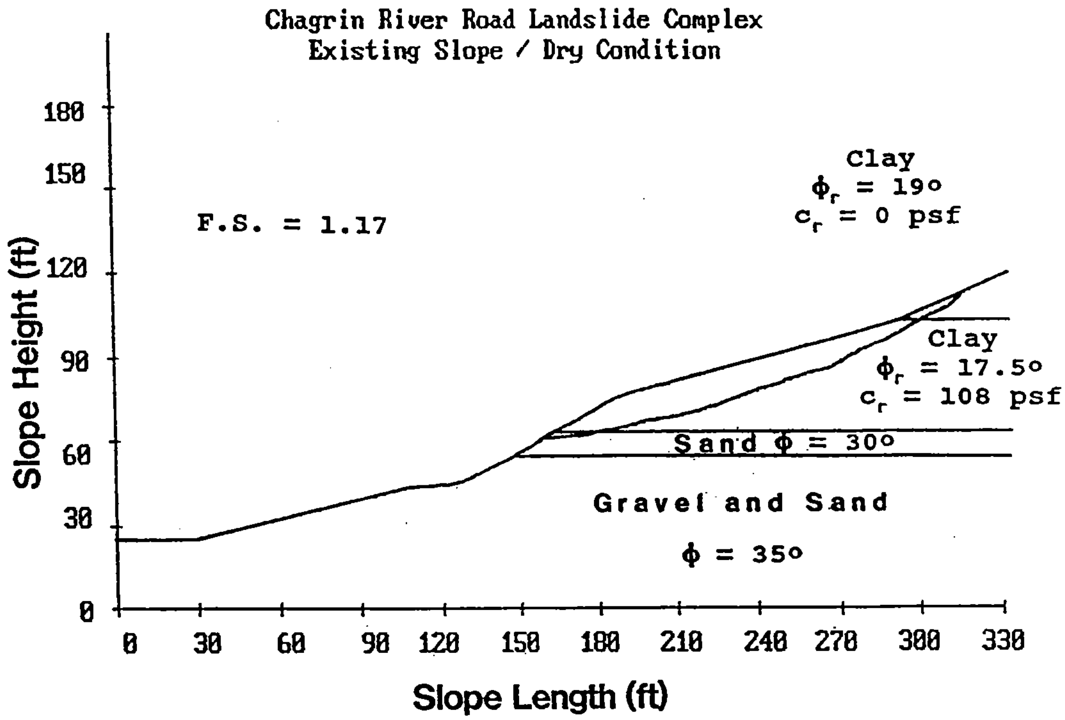


Figure 8: Stability analyses of the original slope for the dry (top) and saturated (bottom) conditions.



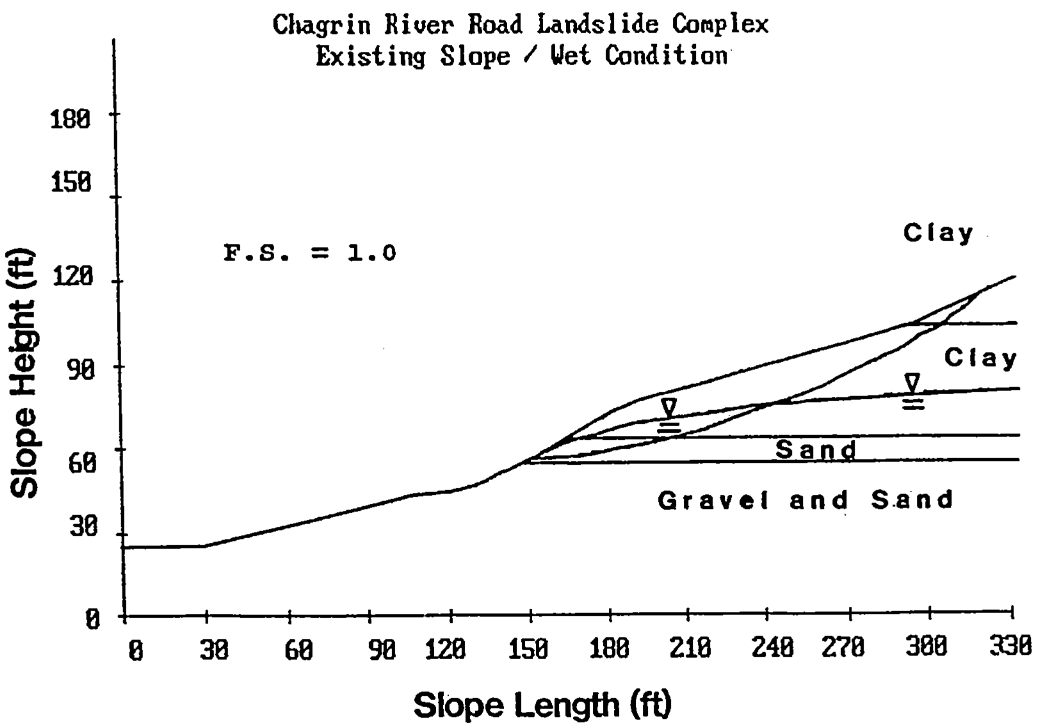


Figure 9: Stability analyses of the existing slope for the dry (top) and saturated (bottom) conditions.

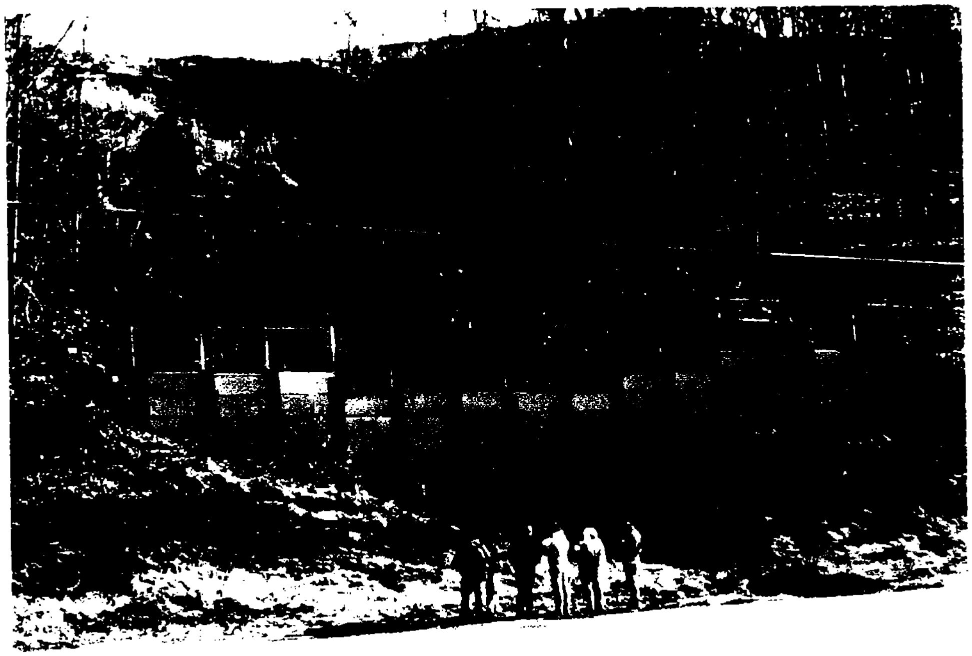


Figure 10: Soldier beam type retaining wall near Pittsburgh, with concrete lagging between the H-piles.

CONCLUSIONS

Results of the study indicate the Chagrin River Road landslide complex is a combination of slump and flow type movements. The stability analyses support the conclusion that water is a significant factor in initiating and perpetuating the failure. Human activity has accentuated the slope movement through oversteepening of the slope and cutting of the toe during the construction of Chagrin River Road, and overloading the top of the slope with a housing development.

The use of improper corrective measures to remediate the slide has unfortunately left the existing slope vulnerable to future slope movements. These measures included removal of slide material from the toe area and poor maintenance of the subdrain system installed to relieve groundwater pressure from the slope. These activities, in conjunction with the weak soils comprising this slope, result in repeated failures at this site.

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Potential for Slope Instability and Acidic Runoff along a section of the US 19 Corridor; Cherokee, Graham, and Swain Counties, North Carolina

B.C. Reed, W.T. McConnell, & D.M. Mullen; NCDOT

Abstract

A four-lane route from Andrews to Almond in western North Carolina has been proposed for nearly thirty years. Environmental, geological, and economic considerations have initiated the A-9 project, which will reroute US 19 north through Graham and Swain Counties around the scenic Nantahala Gorge. The project requires upgrading and connecting existing state routes via cross country alignments through some of the state's most rugged and environmentally sensitive terrain. Numerous large roadcuts and two tunnels will be excavated in medium to low–grade metasediments of the Murphy and Blue Ridge Belts. A thorough geological and geotechnical investigation is underway to assist slope design and to assess the potential for acidic runoff along the corridor.

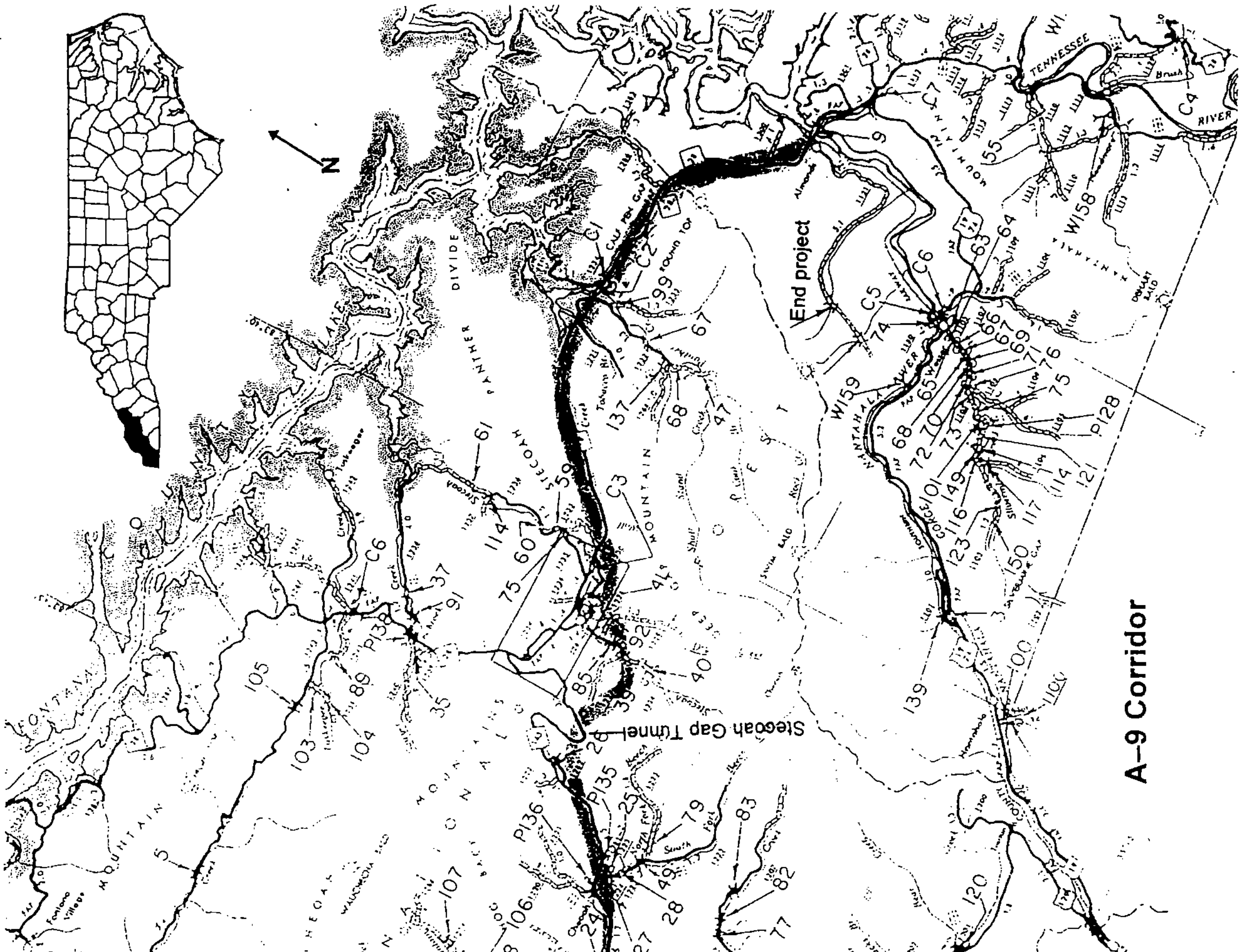
The A-9D corridor crosses the axis of the Murphy Syncline; one of many NE–SW trending orogenic features in the region. Structural mapping in the eastern corridor along NC 28 has identified two sub-orthogonal structural geometries related to 1) initial bedding–foliation formation and 2) later NW–SE compression and thrust faulting. Joints sub–parallel to topography occur locally in association with rebound as overburden is eroded. Relict bedding generally dips south toward the road along much of the alignment. Joints often form release surfaces orthogonal to bedding that make south facing cuts potentially unstable. Several small slope instabilities have occurred along existing NC 28. All proposed cuts require a thorough evaluation of slope stability.

With the aid of geophysical mapping, the proposed corridor alignment minimizes the potential for acidic runoff. A conspicuous bend in the proposed corridor alignment west of Stecoah avoids sulfide–rich rocks of the Wehutty Formation. Chemical analyses of rock samples collected along the corridor were mildly acidic and range from 0% to 22% sulfide and net acid-neutralizationpotential from –0.03 to 9.90. Surface water samples collected from 69 sites along the project have no adverse acidity or geochemistry, and monitoring will continue through construction.

Project History

The Appalachian Regional Development Act of 1965 was intended to bolster the economies of Appalachian communities. Such communities have long been isolated by an inadequate system of narrow, winding two-lane highways. A primary concern is the need to improve transportation in the region. Congress subsequently passed the Appalachian Development Highway System which included a four lane highway, designated Corridor K, across a section of mountains between Asheville, North Carolina and Chattanooga, Tennessee. The corridor includes over 80 miles on US 19 and US 19A in the State of North Carolina.

Although much of the highway is complete, construction has yet to begin along the A-9 Corridor between Almond and Andrews, North Carolina. The NC DOT began examining alternate routes between Andrews and Almond during the summer of 1977. The Scenic Nantahala Gorge was a primary obstacle. US route 19 follows the Nantahala River through the gorge between Almond and Topton. Proposals to upgrade this section of road were met with opposition from environmentalists as well as geologists and engineers who realized that the gorge could not accommodate the highway for environmental and engineering reasons. A total of eight alternatives were considered, but the number soon dropped to four when those involving the Nantahala Gorge were abandoned. By late 1979,



A–9 Corridor
Stecoah Gap Tunnel
Tathum Gap Tunnel
End project
Begin Project
FONTANA
TENNESSEE
RIVER
NANTAHALA RIVER
GORGE
ANDREWS
STECOAH
MOUNTAIN
DIVIDE
PANTHER

the decision was made to reroute the corridor westward from Almond through Swain and Graham Counties to Robbinsville and then south through northern Cherokee County where it rejoins US 19 at Andrews. The new highway will improve traffic flow through these counties, alleviate traffic congestion in the Nantahala Gorge, and provide an improved southwest connection to the town of Robbinsville.

Detailed mapping of rock structures is being conducted to assist slope design along the corridor. The mapping effort includes collecting and testing rock samples for sulfide content and net acid-neutralization potential to identify potential acidic runoff hazards. Water quality in streams along the corridor is being monitored as an on-going supplement to the mapping effort. The following paper discusses the results of mapping completed in the eastern portion of the corridor, A-9D, and the geochemistry of surface water and rock samples collected in the corridor.

Project Corridor

The A-9 project refers to approximately 26 miles of proposed scenic roadway between Almond and Andrews, North Carolina. Much of the project requires upgrading and straightening preexisting roads and widening many cuts to meet the criteria for a four-lane divided highway with a design speed of 55 miles per hour. Where the alignment leaves the preexisting roadway, numerous large roadcuts and two proposed tunnels require a thorough investigation into slope stability and acidic runoff potential. About eighteen miles of the proposed corridor alignment coincide closely with existing roads.

From Almond, the corridor follows NC 28 west to Stecoah where it then curves south of the village for several miles to join NC 143 west of Stecoah Gap. The Appalachian Trail crosses the alignment at Stecoah Gap. A two-lane, single bore tunnel is proposed 550 ft. below the trail. The tunnel will extend 2900 ft. through the mountain along a 1.69% grade. West of Stecoah Gap, the corridor alignment parallels Sweetwater Creek and sections of NC 143 to Robbinsville. East of Robbinsville, the alignment swings south and west along Long Creek and up the north slope of the Snowbird Mountains. The corridor crosses the Snowbird Mountains at Tathum Gap, between Andrews and Robbinsville, where grade restraints, aesthetics, and the potential for disturbing sulfide-rich rocks make tunneling an attractive option. The proposed mile-long tunnel follows a 2.5% grade some 1000 ft. beneath Tathum Gap. The alignment continues down the south slope along Forest Service Rd. 1110 to join the US 19 bypass at Andrews.

Physiography

Much of the A-9 project is within the Nantahala National Forest which contains some very scenic and rugged terrain. Elevations in the project area range from 1875 ft. at Andrews to almost 5000 ft. on nearby balds. Natural slopes up to 30 degrees are common along the corridor. Development is confined primarily to the narrow flood plains of stream and river valleys. Soils are either residual or transported and vary considerably in thickness. Residual soils are the most abundant soil type. Colluvium is the most common transported soil and alluvial soils are confined to stream valleys. Topography strongly influences local precipitation and ecology. South and west facing slopes receive more sunlight and are usually warmer and drier than north and east slopes. The region is typically humid with a mean annual rainfall of 60 inches at Robbinsville. The project area is heavily forested and supports a diverse assemblage of plant life and plant community types depending on topography, sunlight, precipitation, and soil . The following plant communities were mentioned in the 1984 Environmental Impact Statement for the A-9 Corridor:

Cove forests are composed of hardwood trees and shrubs that occupy lower slopes, valleys, and sheltered, north facing hollows. They often contain a diverse herbaceous flora as well as ferns, mushrooms, and mosses.

Hemlock forests are common in acidic soils from 2000 to 4500 feet. Hemlock stands are mixed with hardwoods at lower elevations and commonly have an understory of rhododendron but little undergrowth.

Oak and Hickory forests are the predominate hardwood stands. White and red oak are more common on cool moist slopes while chestnut, black, and scarlet oak flourish on dry south or west facing slopes.

Pine forests, either pure stands or mixed with hardwoods, commonly occupy drier sites.

River margin stands comprised of sycamore, ash, basswood, poplar, beech, maple, and dogwood are confined to narrow stream valleys.

Succession communities are quick growth-short-lived plants that occupy open areas or recently disturbed soils such as utility right-of-ways, roadsides, and abandoned fields.

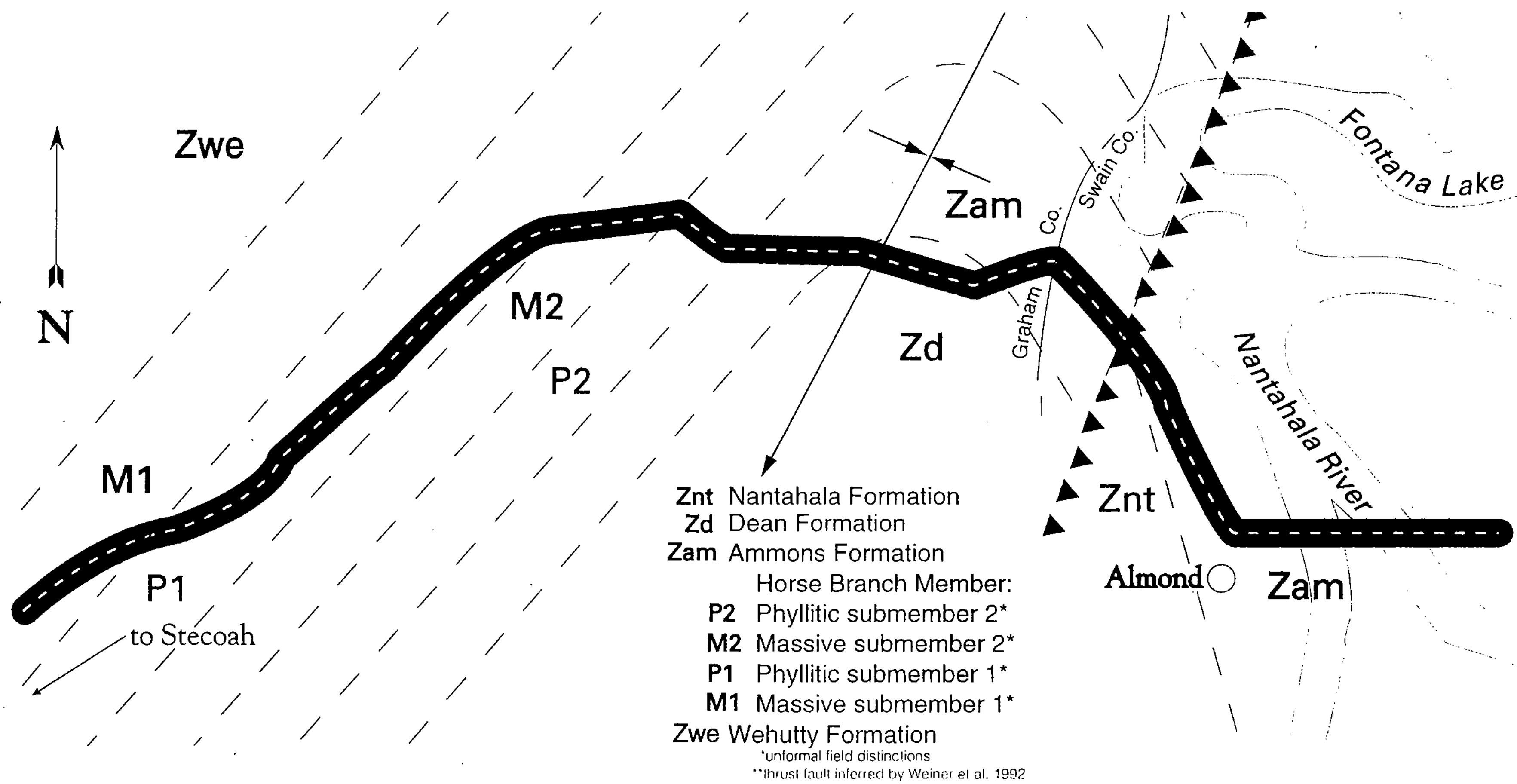
Intermittent streams dissect the uplands and empty into more permanent streams in the valleys. Most of the project lies within the Tennessee River Basin. The Cheoah Mountains form a primary east-west drainage divide. North flowing streams flow directly into the Tennessee River, or Fontana Lake above the dam, while south and east flowing streams drain into the Nantahala and Cheoah Rivers before emptying into the Tennessee River. The Snowbird Mountains form another drainage divide in the southern portion of the project. Streams flowing north from the Snowbirds empty into the Cheoah River and south flowing streams empty into the Valley River which drains into the Hiwassee River Basin. Mean annual temperatures in the project area range from 38°F to 70°F. Frost wedging during colder periods degrades rock stability, particularly in the higher elevations. Because of abundant precipitation, steep slopes, and fluctuating temperatures; landslides are not uncommon in the region especially when combined with adverse geology. However, abundant vegetation on most slopes greatly reduces the number of slides.

Geology

The rocks underlying the A-9 project are low to medium grade metasediments of the Late Proterozoic Murphy and Blue Ridge Belts. For a complete description of these units, refer to the Geologic Map of southwestern North Carolina including adjoining southeastern Tennessee and northern Georgia (Wiener & Merschat 1992). The Murphy and Blue Ridge Belts are the result of several episodes of deformation during Appalachian Orogenies which accounts for their metamorphism. In much of the region, the foliation coincides with relict structures such as bedding planes and dips moderately to the southeast about 30 degrees. Textural foliation or preferred planar alignment of minerals often obscures relict structures in some rocks that have undergone higher grades of metamorphism. The metasedimentary rocks are folded to varying degrees regionally and locally. Folding causes local deviations from the southeast dip direction. Continued compression during later orogenic episodes resulted in displacement along enormous thrust faults. A series of local thrust faults uplift and expose many Proterozoic rocks near the project. Large footwall synclines and headwall anticlines are commonly associated with these thrust faults. The northeast trending Blue Ridge Thrust is a prominent structure in the region and the platform for the Blue Ridge Mountains. Numerous other northeast trending, orogenic faults and folds deform the rocks of the Murphy and Blue Ridge Belts. Metamorphic facies range from biotite to staurolite (Butler 1984), although no staurolite has yet been encountered in the eastern corridor. The principal compressive stress throughout the Appalachian Orogeny remained southeast–northwest as the African and North American plates slowly collided. The Murphy and Blue Ridge Belts lie along the western terminus of this collision zone. Prolonged tectonic quiescence and a humid climate have resulted, locally, in deep weathering of bedrock and the formation of abundant saprolite and residual soil.

The Murphy Marble and Brasstown Formations of the Murphy Belt are encountered only in the southern terminus of the project near Andrews. The Nantahala Formation of the Murphy Belt cores the Snowbird Mountains and is a potential acid-runoff hazard. A sliver of the Nantahala Formation outcrops south of US 28 near the Graham and Swain County line as a result of thrusting along the Mary King Mountain Fault. The Dean Formation of the Ocoee Supergroup of the Blue Ridge Belt outcrops just west of the Graham and Swain County line and again on the north slope of the Snowbird Mountains. The following map depicts the units mapped thus far along the project east of Stecoah. The Ammons Formation and its Horse Branch Member underlie most of the corridor. Because of its predominance along the A-9D corridor, the Horse Branch Member of the Ammons Formation is divided for engineering purposes into four submembers based upon spacing and orientation of discontinuities:

Geology and Slope Stability along the A-9 D Corridor from Almond to Stecoah



Massive Submember #1 (M1)- Slightly foliated feldspathic gray quartzite with widely spaced joint sets.

Massive Submember #2 (M2)- Blue to gray feldspathic quartzite with highly variable bedding thickness from 4 inches to 6 feet.

Phyllitic Submember #1 (P1)- Thinly foliated tan to gray arkosic, quartz-rich, phyllitic muscovite schist. Commonly exhibits red oxidized coating on weathered surfaces.

Phyllitic Submember #2 (P2)- Thinly foliated tan to gray quartz rich, muscovite and biotite phyllite containing thin dark gray bands.

A generalized model of the structural geometry for the region is based on southeast–northwest compression. The model includes two primary relationships: 1) compressive features orthogonal to tensile features and 2) strike-slip or thrust features acute to principal compression. Compression is depicted by northeast–southwest trending folds such as the Murphy Syncline, huge southeast dipping thrust faults such as the Mary King Mountain Fault, and to a lesser extent, northwest dipping back-thrusts. West to northwest striking transform faults offset the great thrust ranges to accommodate displacement along the earth's curved surface. Tensile features include northwest–dipping normal faults or joints, northwest–striking vertical joints, and exfoliation joints. Each orogenic episode has its own associated strain regime which may become superimposed upon the previous event. Preferentially oriented, older structures may also accommodate some newer stresses.

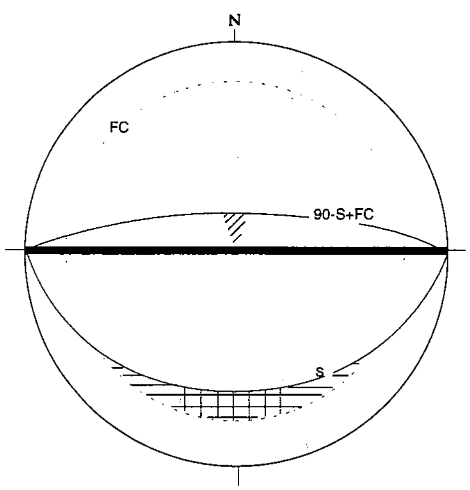
Localized seismic activity continues in the southern Appalachians, and the reactivating of preexisting faults is suspected (Sibol et al. 1994). Changes in current stress patterns and isostatic rebound are believed to be source of this seismicity and may be accommodated along preferentially oriented discontinuities. The energy released during a typical seismic event in the area appears to be insufficient to warrant major construction measures, however, a historical record of seismic data is being examined and included in the design of roadway structures and embankments. Faults encountered along the project are closely examined for evidence of recent movement and the sense and amount of ancient displacement.

Potential for Slope Instability

Detailed geological mapping and sampling have been completed along NC 28 between Almond and Edward's Gap about 0.5 miles east of Stecoah, the section referred to as A-9D. Much data were collected from the numerous outcrops along the highway and feeder roads. Foliation planes coincide with bedding surfaces and are thought to be the primary controlling factor in the stability of most slopes. Most of the highway lies on rocks of the Ammons Formation and its Horse Branch Member. Further divisions of the Horse Branch Member, as discussed above, were necessary to distinguish units based on engineering criteria. Whenever practical, a combination line–window mapping method is used to obtain statistically unbiased data from each roadside outcrop. At approximately four feet above the ground, a tape is stretched from which dip and dip direction measurements are taken at two foot intervals. A window of two feet above or below the tape usually maintains the constant sampling interval. If possible, measurements are also taken from the top of outcrops, around bends in the cut face at angles to the road, or along feeder roads to prevent under sampling of high-angle features oriented sub–parallel to the alignment. Structural features are divided into four groups: 1) ***Bedding or Foliation.*** 2) ***Joints on foliation.*** Because foliation tends to form planes of weakness, joints often develop along preferentially oriented foliations. Such features in highly folded rocks may utilize multiple foliation planes if they can best propagate the joint. These joints may be the result of exfoliation during rebound as overburden is removed or even shear planes accompanying thrust faulting. 3) ***Joints.*** These include joints not parallel to foliation, fractures, and faults. 4) ***Zones***. These include major features with closely spaced fractures such as shear zones where deformation was not limited to a single plane.

Structure type, orientation, rock type, and outcrop number are numerically coded and recorded in the field on a palm-top computer. Outcrop locations and station numbers are recorded. The rock structure data are downloaded onto an office PC and converted to a format compatible with engineering stereonet programs. Pole plots of discontinuities are produced for each outcrop. All the pole plots are analyzed spatially to determine general trends along the corridor. Stereonets are then compiled for each cut using data from adjacent outcrops.

A simplistic slope evaluation is made using stability nets like the one below to screen slopes that contain features which may daylight within wedge, plane, and topple zones. Dip vectors and wedge intersections are plotted in relation to roadcut orientation, proposed slope angles, and an estimated or determined friction cone. In the absence of measured friction values, a conservative friction cone of 20 degrees is used for these nets, and slopes of 0.25:1 and 1.5:1 represent the proposed range of roadcut angles. These slope values are also complimentary topple limits. Wedge intersections are approximated from pole plots by 1) clustering joints and foliations, 2) plotting planes to cluster centroids, and 3) identifying general orientations of the intersections of these planes. Wedge intersections behave as vectors by controlling the direction in which failed wedges slide out of the slope face. This method only identifies those slopes which will require in-depth analysis. The final slope designs will be based on a rigorous probabilistic analysis as processed through the NCDOT decision analysis framework, a discussion of which is beyond the scope of this paper.



Dip Vector Plot of Daylight Zones for a South-Dipping Roadcut with a 1 1/2 : 1 Slope

FC = Friction Cone S = Cut-Slope

Planar failure Topple failure Wedge failure

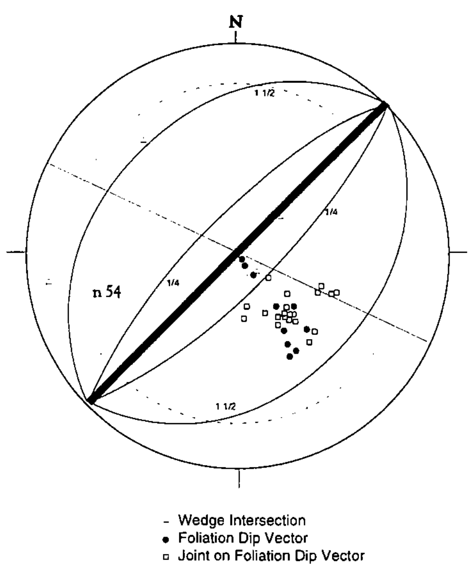
The A-9D project crosses the axis of the southwest trending Murphy Syncline. The Murphy Syncline is disrupted by the Mary King Mountain Fault south of the corridor. The fault is inferred across the corridor near the Graham and Swain County line. Bedding data from A-9D define the Murphy Syncline in detail. The outcrop data show the change in bedding orientation from a predominately southwest dip near Almond to a predominately southeast dip in Graham County. The syncline axis crosses beneath the corridor near the Graham and Swain County line. Data from the Dean formation depict a bimodal band of south dipping beds expected across the nose of a fold. Crenulations are present in some schistose zones and trend roughly parallel to the axis of the Murphy Syncline. Most outcrop data display a group of nearly vertical, northwest striking joints. Some of these joints are orthogonal to bedding, but most are likely related to the thrusting event and occur on both limbs of the Murphy Syncline. Northwest dipping tensile features, orthogonal to bedding, are generally lacking overall. Some northwest dipping discontinuities occur locally and are perpendicular to southeast dipping foliations.

Foliation planes along A-9D generally dip south toward the road. Joints often form release surfaces orthogonal to bedding that make south facing cuts potentially unstable as depicted in the following figures, particularly when joints are developed along bedding. The present 1:1 cut slope along this section of NC 28 allows joints on bedding to daylight. Exfoliation joints occur locally in association with rebound as overburden is eroded, contributing to potential planar instability. Planar instability is a major concern primarily for southeast-facing cuts, and one stretch of southwest facing cuts near the thrust fault. The southwest-facing cuts are in the Ammons Formation on the east limb of the Murphy Syncline where beds dip from 30° to 40° southwest. North facing cuts along this section of the corridor are less susceptible to planar instabilities. Most topple zones within the stability nets for cuts along A-9D are void, however, northwest facing cuts in the M1 and P1 units near Stecoah are potentially susceptible to toppling. A detailed analysis of fracture spacing, length, and

Dip Slip Instability in Ammons Formation along NC 28.

roughness is being performed along this section of the corridor to assess the probability of toppling. Wedge instability is a potential problem for all cuts and is more important than planar instability when bedding strikes obliquely to the roadway. Slopes flatter than 1:1 eliminate many candidates for wedge instabilities. If high angle slopes are incorporated into the design along the A-9D section of the corridor, then rockfall mitigation measures such as ditches and fences will be necessary to mitigate wedge instabilities on larger cuts.

Daylight Net for NC 28 rockslide.



Acid Runoff Potential

Many metasedimentary units in the southern Appalachians are composed of pyritic rocks. Excavation and exposure of these rocks facilitates the generation of acid by weathering. Disseminated, fine-grained sulfides in less resistant units are of particular concern because they are rapidly reduced and released into the water cycle. Numerous reports link the degradation of surface waters and fish kills to mining and road construction in the region. In the late 1970's acid-related problems were encountered in the construction of a scenic–forest highway west of the A-9 project. The use of pyrite-rich fill material resulted in the leaching of iron sulfide into nearby trout streams, severely raising the acidity (Byerly & Middleton 1981). A similar problem was encountered near the junction of US 19 and NC 28 east of the A-9 project. Although mitigation of that site proved problematic, a series of wetland retention ponds designed by Dr. Don Byerly of the University of Tennessee appears to be stabilizing acidity. Samples collected from the ponds maintain pH levels well above a the acidity limit of 4.5

Based on concern over acidic runoff, an induced polarization (I.P.) survey was conducted along the A-9 corridor in the summer of 1980. I.P. is a nondestructive geophysical method that exploits the ability of subsurface materials to conduct an electrical current via ions in solutions filling the interstices of the rock. The presence of metallic minerals such as pyrite can create a blocking action or an induced polarization where conduction changes from ionic to metallic at the solution-mineral interface (Phoenix Inc. 1980). Many of the rocks along the corridor contain highly disseminated, minute pyrite or marcasite crystals that are not readily identifiable by other geophysical methods. Rocks comprised of more than 0.5% pyrite are suspected of producing acid runoff. I.P. surveys can provide an estimate of the concentration (as low as 1%) and spatial distribution of metallic minerals disseminated in the subsurface (Phoenix Inc. 1980).

The I.P. method cannot distinguish between iron sulfides and other metallic minerals, nor can it detect minerals in the rock that behave as buffers to acid runoff. Therefore, an acid-base-accounting test is necessary to determine a net neutralization potential (NNP) which is simply the difference between its neutralization potential or measure of bases present (NP) and its acid potential or amount of acid present (AP). Rock's with NNP values below –5 (NP-AP) are considered "hot" and are avoided if possible or encapsulated in an approved waste site: Rocks with NNP values above 0 are considered "cold" and can be used as general embankment or crushed–aggregate material. Rock excavations having NNP values between –5 and 0 are considered "warm" and are treated with lime and placed in controlled embankments or waste sites.

The I.P survey identified the Nantahala Formation as a potential pyritic-rich unit. Subsequent acid-base testing of drill cores from the Nantahala Formation north of Tathum Gap confirm a high acid runoff potential (NNP <–5) and suggest that the unit be avoided where possible. Most of the sulfide material appears to be a near-surface occurrence related to weathering. Alignment changes are being considered between Andrews and Robbinsville to avoid potentially high concentrations of sulfides in the Nantahala Formation. The Wehutty Formation contains sulfidic schist similar to the rocks encountered along the forest highway mentioned above. As a precaution, the alignment curves around Wehutty rocks west of Stecoah.

A screening for rock quality investigation was done along the A-9D corridor and is underway along the rest of the corridor. Representative rock samples were collected from outcrops east of Stecoah: Samples were taken from the Ammons Formation, the Dean Formation, and the Horse Branch Member of the Ammons Formation. The samples were sent to the University of Tennessee, Knoxville where NNP values were determined. Pyrite cubes were visible in the metasandstone layers of the Ammons Formation and minute pyrite grains were suspected to be disseminated in the phyllitic submembers P1 and P2. The test results listed below suggest that the acid runoff potential along the eastern corridor is minimal, however tests will be performed on cores from all proposed cuts. Because of the complexity and unpredictability of sulfide

Acidic Potential, Rock Samples A-9D

NC DOT 8/18/94	STATION	N HCl/NOH	Amt. HCl	Amt. NaOH	C	Amt. NaOH	Acid	NP	%S	AP	NP-AP
SAMPLE ID	#	Used	Used	Sample		Sample	Consumed				(NNP)
A9DC1	0+847-0+892	0.10	20.00	20.13	0.99	17.30	2.81	7.03	0.00	0.13	6.90
A9DC2	2+930-2+968	0.10	20.00	20.13	0.99	19.50	0.63	1.56	0.00	0.12	1.44
A9DC3	4+405-4+548	0.10	20.00	20.13	0.99	18.60	1.52	3.80	0.01	0.16	3.64
A9DC4	5+126-5+198	0.10	20.00	20.13	0.99	17.50	2.61	6.53	0.00	0.10	6.43
A9DC5	5+973-6+116	0.10	20.00	20.13	0.99	19.60	0.53	1.32	0.00	0.15	1.17
A9DB6	6+428	0.10	20.00	20.13	0.99	17.90	2.22	5.54	0.00	0.15	5.93
A9DB7	6+943-7+145	0.10	20.00	20.13	0.99	19.30	0.82	2.06	0.01	0.33	1.73
A9DB8	7+877-7+935	0.10	20.00	20.13	0.99	17.50	2.61	6.53	0.09	2.72	3.82
A9DB9	9+050-9+105	0.10	20.00	20.13	0.99	20.00	0.13	0.32	0.00	0.12	0.20
A9DB10	9+953-9+997	0.10	20.00	20.13	0.99	17.40	2.71	6.78	0.22	6.81	-0.03
A9DB11	11+275-11+367	0.10	20.00	20.13	0.99	17.40	3.01	7.53	0.20	6.31	1.21
A9DB12	Y14 1+100	0.10	40.00	40.30	0.99	36.30	3.97	9.93	0.00	0.03	9.90

deposits in the area, samples from all rock excavations made during construction will be thoroughly examined for acid runoff potential.

A baseline study of all springs and streams encountered along the A-9 project is underway to assess existing pre-construction water quality to help determine the level of any degradation caused by construction and the efficacy of mitigation measures. Water sampling sites are chosen just outside the construction limits on the basis of accessibility before, during, and after construction. A number of stream channels within construction limits will have to be relocated or piped through fill material. If a drainage feature traverses the project, samples are taken above and below the proposed corridor. A water sampling and testing program developed for A-9D utilizes 69 sites. All 69 sites were sampled and tested initially in August 1994. Seventeen of the sites have been tested monthly since June 1994. Monthly testing of the other 52 sites will begin as their respective project areas near construction. Some mean results of water testing to date are shown below.

Water temperature and pH are measured in the field using a standard thermometer and a portable pH meter. Samples are collected at each site and sent to a commercial laboratory where they are tested using EPA standards for conductivity, alkalinity, pH, acidity, sulfate, iron, manganese, and aluminum. Additional testing for copper, zinc, cadmium, and chromium may be performed on anomalous samples. Tests for turbidity and total suspended solids are being established and will involve the installation of automated devices to

Water Quality A-9D (March 1994-June 1995)

Sample No.:	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
pH	7.38	7.26	7.18	7.97	7.34	7.47	5.91	7.30	7.13	7.41	7.20	7.39	7.40	7.25	7.39	7.11	7.48
Cond. (umhos)	65.7	141.1	59.3	137.3	97.2	72.4	47.7	20.5	19.3	17.5	19.2	32.0	27.1	118.2	30.6	187.1	26.4
Sulfate (mg/l)	2.9	2.3	2.5	1.8	3.1	2.5	8.4	1.7	1.3	1.2	1.3	1.3	1.5	30.9	2.4	47.7	2.7
Al (mg/l)	0.306	0.102	0.218	2.671	0.367	0.310	0.132	0.850	0.761	1.276	1.033	0.729	0.457	0.696	0.428	0.819	0.672
Fe (mg/l)	0.773	0.605	0.556	1.983	0.675	0.641	0.623	1.231	1.131	0.743	1.068	0.941	0.544	1.649	0.731	2.246	0.664
Mn (mg/l)	0.113	0.309	0.191	0.181	0.466	0.197	0.289	0.074	0.040	0.075	0.039	0.066	0.030	0.956	0.031	1.732	0.030
Alkalinity	19.5	59.2	16.6	66.9	36.5	23.1	11.2	10.0	8.4	7.9	9.7	13.9	12.7	13.7	10.0	21.1	7.5
Acidity	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1

sample runoff during storm events. Water quality along the eastern corridor, A-9D, appears to show no adverse acidity or geochemical properties, however, the NCDOT will continue to monitor water samples through construction.

CONCLUSIONS

The proposed A-9 corridor is not ideal from a geological perspective. The project lies within some of the most scenic and rugged wilderness in the Southeast. Some degradation of stream quality and habitat destruction is inevitable during construction and the aesthetic impacts will be permanent. Geological and geotechnical investigations along the eastern corridor, A-9D, suggest that there is a definite potential for slope instability and acid runoff. All of the rock units on the project contain some pyrite, although some are more likely to cause acid runoff than others. The establishment of sampling and testing methods to monitor rock and water quality as well as the installation of automated, runoff catchments will greatly assist mitigation procedures during construction. South facing slopes are proposed for all sections and are especially vulnerable to planar instability in section D. Alternative design measures such as bridges or viaducts may be considered to avoid cuts in potentially unstable or "hot" rocks and to reduce earthwork quantities. Such bridges have been used to greatly reduce the number of and lengths of tunnels on other major highway projects in the United States and abroad. Retaining walls may also serve to keep fill material from encroaching on nearby streams. These roadway structures also minimize the need for stream channel changes and limit the barrier effect that conventional highway designs pose to migrating wildlife. Carefully planned design parameters incorporating background geological data and environmental mitigation measures will limit the amount of degradation caused by construction and the amount of future maintenance required to upkeep the roadway.

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Landsliding in Pennsylvania

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SYNOPSIS Pennsylvania, a state in the eastern United States, is well known for landsliding. Much of Pennsylvania lies in the Appalachian Plateaus, one of the major areas of landslide severity in the United States. Deep-seated rock slides are rare in Pennsylvania but rock falls and topples are common. Soil landslides are most common in colluvium and fill, especially in the Appalachian Plateaus. Heavy precipitation is a common triggering mechanism for landslides. Man's construction activities have also caused many landslides in Pennsylvania. Cost data on landslide damages are limited but these damages are known to be economically and socially significant. Proven technology is available for minimizing adverse effects of landslides and some municipalities have promulgated grading codes in this regard. Rigorous review, inspection, and enforcement are essential if these grading codes are to achieve their purpose of reducing landslide damages.

INTRODUCTION

Pennsylvania (Fig. 1) has long been known as a state with significant landsliding. This results from Pennsylvania's humid temperate climate, varied geology, locally steep and rugged topography, and long history of man's activities. The latter include those related to transportation; industrial, commercial, and residential development; and mining, particularly coal mining. Landslides have occurred in many parts of Pennsylvania but they are most abundant and most troublesome in the Appalachian Plateaus of western and northern Pennsylvania (Figs. 1 and 2).

The Appalachian Plateaus, including those of Pennsylvania, are one of the major areas of landslide severity in the United States (Baker and Chieruzzi, 1959; Gray, 1977; Gray, Ferguson, Hamel, 1979; Hamel, 1980; Pomeroy, 1982; Radbruch-Hall, et al., 1982). Within the Appalachian Plateaus, the Monongahela River valley of northern West Virginia and southwestern Pennsylvania (extending south from Pittsburgh, Fig. 2) has a special place in landslide folklore. "Monongahela" is derived from the American Indian name for the river which is translated as "river with the sliding banks" or "high banks which break off and fall down" (Espenshade, 1925).

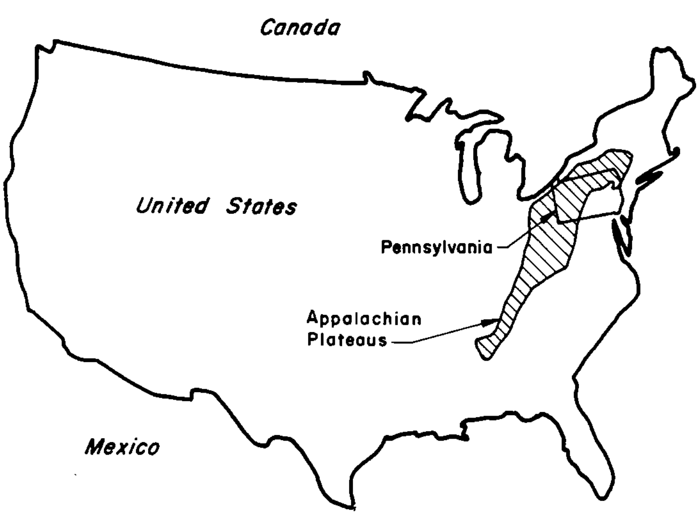

Fig. 1 Location of Pennsylvania

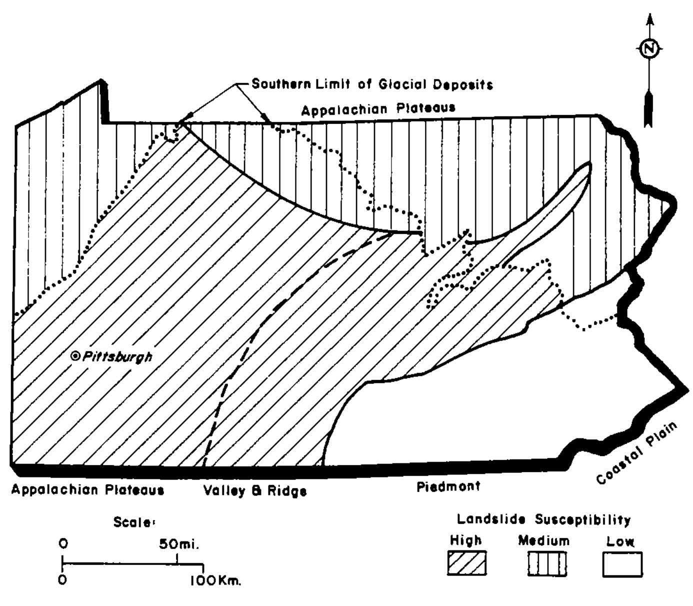

Fig. 2 Landslide susceptibility in Pennsylvania

This paper is a very brief overview of landsliding in Pennsylvania. Areas of relative landslide susceptibility are outlined. Landslide types and processes in both rock and soil are described. Causes and effects of landslides are discussed and some thoughts on landslide investigation and treatment are presented.

GEOLOGY AND LANDSLIDE SUSCEPTIBILITY

The geology of Pennsylvania is treated in a forthcoming book (Shultz, 1985) and the geology of western Pennsylvania, as related to landsliding, is summarized by Gray, 1977; Gray, Ferguson, Hamel, 1979; Hamel, 1980; and Pomeroy, 1982. The western and northern parts of Pennsylvania are in the Appalachian Plateaus Physiographic Province. These plateaus consist of flat-lying sedimentary rocks of Paleozoic age. The Appalachian Plateaus in northeastern and northwestern Pennsylvania were covered by continental ice sheets during the Pleistocene Epoch (Fig. 2). Portions of Pennsylvania south of the glacial boundary were significantly affected by the ice sheets, e.g., damming and ponding of rivers by glacial ice and/or soil, accelerated erosion of river valleys in bedrock, deposition of outwash sediments in some of these valleys, and accelerated rates of weathering and mass wasting due to ~~the periglac~~ial climate.

The Appalachian Plateaus are bounded on the east by folded Paleozoic age sedimentary rocks of the Valley and Ridge Physiographic Province (Fig. 2). Southeastern Pennsylvania, which includes portions of the Piedmont and Coastal Plain Physiographic Provinces, has bedrock ranging from Cenozoic to Pre-Cambrian in age and topography generally less rugged than the Appalachian Plateaus and Valley and Ridge Provinces to the north and west (Fig. 2).

Zones of landslide susceptibility in Fig. 2 were developed from published information (Baker and Chieruzzi, 1959; Radbruch-Hall, et al., 1982). Southeastern Pennsylvania (primarily the Coastal Plain and Piedmont Physiographic Provinces) has generally low landslide susceptibility. Northwestern and northeastern Pennsylvania (primarily glaciated portions of the Appalachian Plateaus) have medium landslide susceptibility. Most of western and central Pennsylvania (primarily the un-glaciated Appalachian Plateaus and the Valley and Ridge Physiographic Provinces) has high landslide susceptibility.

The zones of landslide susceptibility in Fig. 2 are consistent with topography and geology, with the writers' observations and experience, and with generalized information on natural geologic hazards (Wilshusen, 1979). In view of the site-specific nature of landslides, the importance of human activity in many landslides, and the bias of landslide incidence data toward more heavily populated areas, it seems unlikely that a more detailed map of landslide susceptibility in Pennsylvania is warranted, at least for purposes of this paper.

LANDSLIDE TYPES AND PROCESSES

Deep-seated rock slides (Figs. 3a, 4, 5) are rare in Pennsylvania at present (Gray, Ferguson, Hamel, 1979; Hamel, 1980). Such rock slides are believed to have been more common during Pleistocene time in the Appalachian Plateaus when rivers rapidly entrenched valleys in bedrock and climatic conditions were more rigorous (Ferguson, Hamel, 198 Hamel, Adams, 1981). The best documented recent deep-seated rock slide in the Appalachian Plateaus of Pennsylvania (Fig. 5) occurred at Brilliant Cut in Pittsburgh (Fig. 2) in 1941 (Ackenheil, 1954; Hamel, 1972; Gray, Ferguson, Hamel, 1979). Shallow slab-type rock slides are fairly common at present in steeply dipping strata of the Valley and Ridge Province (Wilshusen, 1979).

Rock falls and topples (Fig. 3d) are much more common than rock slides, particularly in the Appalachian Plateaus and Valley and Ridge Provinces (Ackenheil, 1954; Gray, Ferguson, Hamel, 1979; Wilshusen, 1979). Rock falls and topples occur in both natural and excavated slopes. Weathering and erosion undercut joint-bounded rock blocks which slump backward or topple forward depending on geometry, support conditions, and applied forces which, in addition to gravity, often include water and ice thrusts and root pry (Hamel, Ferguson, 1983). Rock fall volumes are typically

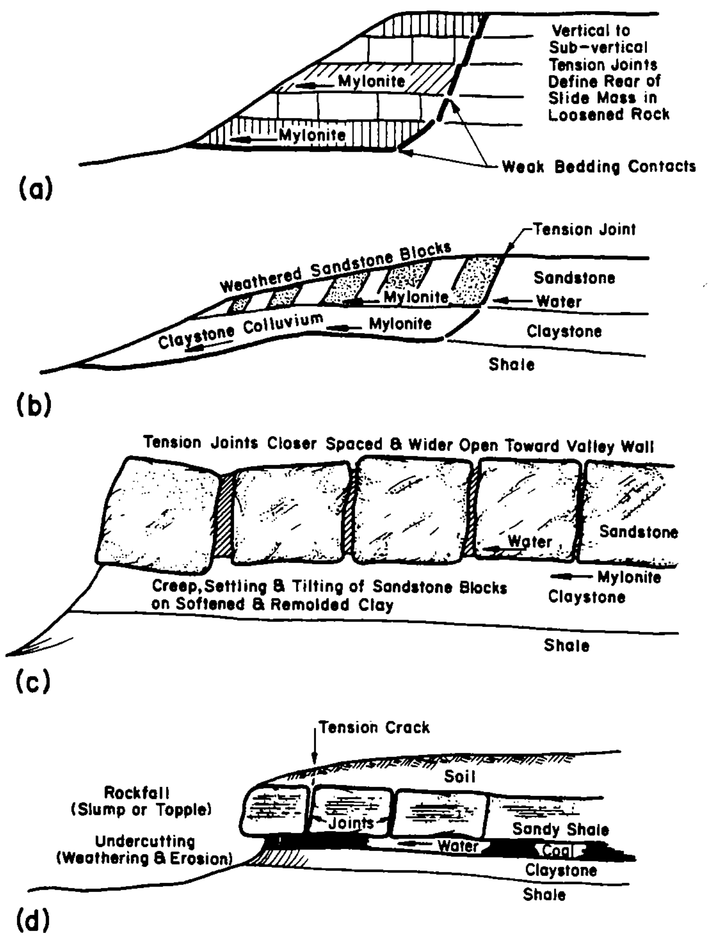

Fig. 3 Valley wall stress relief features in flat-lying sedimentary rocks, e.g., Appalachian Plateaus:
(a) Rock block slide
(b) Colluvium slide
(c) Rock block creep
(d) Rock fall

(After Ferguson, Hamel, 1981)

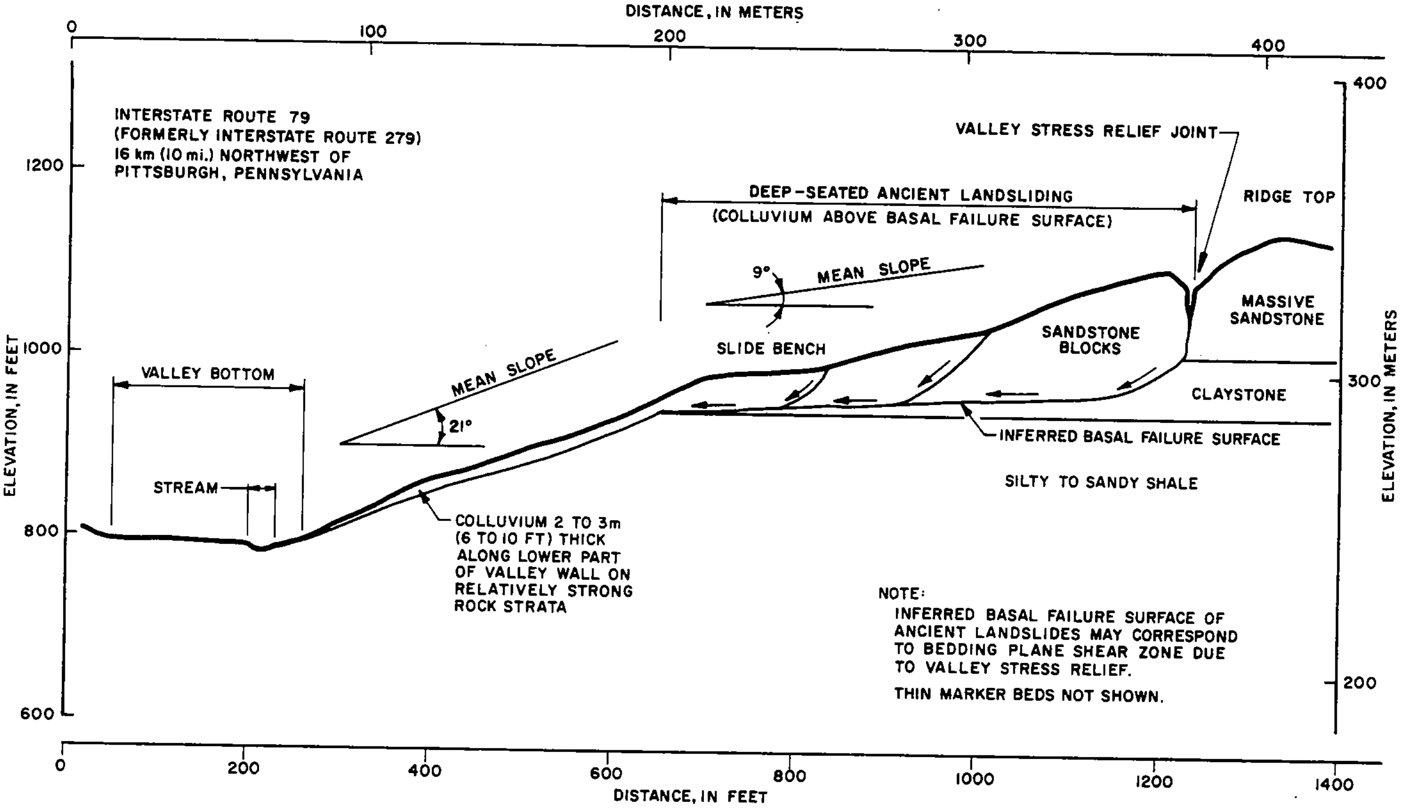

Fig. 4 Cross-section of deep-seated ancient landslide in weak claystone (After Hamel, 1980; Hamel, Adams, 1981)

small, ranging from approximately 0.1 to 100 m^3 (Ackenheil, 1954; Gray, Ferguson, Hamel, 1979).

Soil landslides in Pennsylvania are most common in colluvium (Figs. 3b, 4, 6) and fill (Fig. 7), especially in the Appalachian Plateaus. Some landslides also occur in residual soils of the Piedmont, Valley and Ridge, and Appalachian Plateaus; in the glacial soils of northeastern and northwestern Pennsylvania; and in alluvial soils along major rivers.

Colluvium is old landslide or creep debris. This material is potentially unstable because past landslide or creep movements have reduced strength along one or more surfaces generally parallel or sub-parallel to the slope (Figs. 3b, 4, 6). Colluvium is particularly common in the Appalachian Plateaus of western and northern Pennsylvania (Hamel, 1980). Talus deposits, which are not uncommon in north-central Pennsylvania, can be considered similar to colluvium for purposes of this paper.

Fill is soil and/or rock material placed by man. The stability of fill varies widely depending on material characteristics, placement techniques, and surface and subsurface drainage. Engineered fills are constructed of suitable soil and/or rock materials spread in horizontal layers of appropriate thickness and properly compacted. These fills are typically benched into residual soil and/or weathered rock and provided with appropriate surface and subsurface drainage (Fig. 8). Non-engineered or random fills lack some or all of the features of engineered fills and are often simply dumped or pushed over a hillside.

Fill slides are particularly common in the Appalachian Plateaus as a result of this region's steep topography; unfavorable geology including weak rock strata, colluvium, and springs; and coal mining with related spoil and refuse placement. This region also has a propensity for non-engineered fill which apparently reflects an Appalachian heritage of throwing things over hillsides.

Landslide processes in Pennsylvania soils include slides, avalanches, flows, falls, and topples (Schuster, Krizek, 1978; Chap. 2). Slides occur in all soils but are most common in colluvium (natural or excavated slopes) and fill (man-made slopes, often underlain by or adjacent to colluvium). Most soil slides in Pennsylvania have relatively slow movement rates. These slides include slumps with rotational movements, block slides with translational movements, and combinations of slumps and block slides.

Avalanches and flows occur in most soils but are especially common in colluvium and fill. Some earthflows occur in glacial and alluvial soils (Hamel, 1983, 1985). Avalanches and flows are frequently associated with heavy rainfall and/or rapid rise in groundwater level (Gray, Ferguson, Hamel, 1979; Hamel, 1980, 1983, 1985; Pomeroy, 1980, 1982, 1984). Movement rates range from slow for some earthflows to very rapid for other earthflows and most avalanches. The lower por-

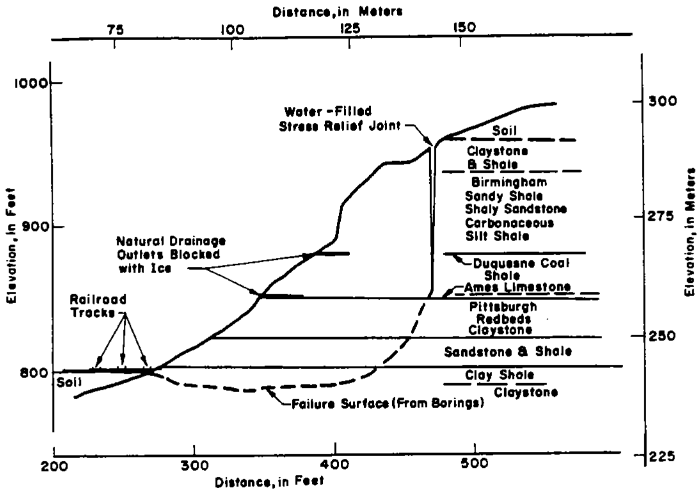

Fig. 5 Cross-section of 1941 rock slide at Brilliant Cut, Pittsburgh
(After Ackenheil, 1954; Hamel, 1972; Gray, Ferguson, Hamel, 1979)

tions of soil slide masses often disintegrate into avalanches or flows during periods of heavy rainfall or rapid groundwater rise (Fig. 7).

Soil falls and topples are similar to rock falls and topples. A soil slab or block slumps backward or topples forward depending on geometry, support, and loading conditions. These movements occur rapidly in excavated slopes in all soil types and in oversteepened natural slopes, e.g., stream banks in glacial or alluvial soils (Hamel, 1983, 1985).

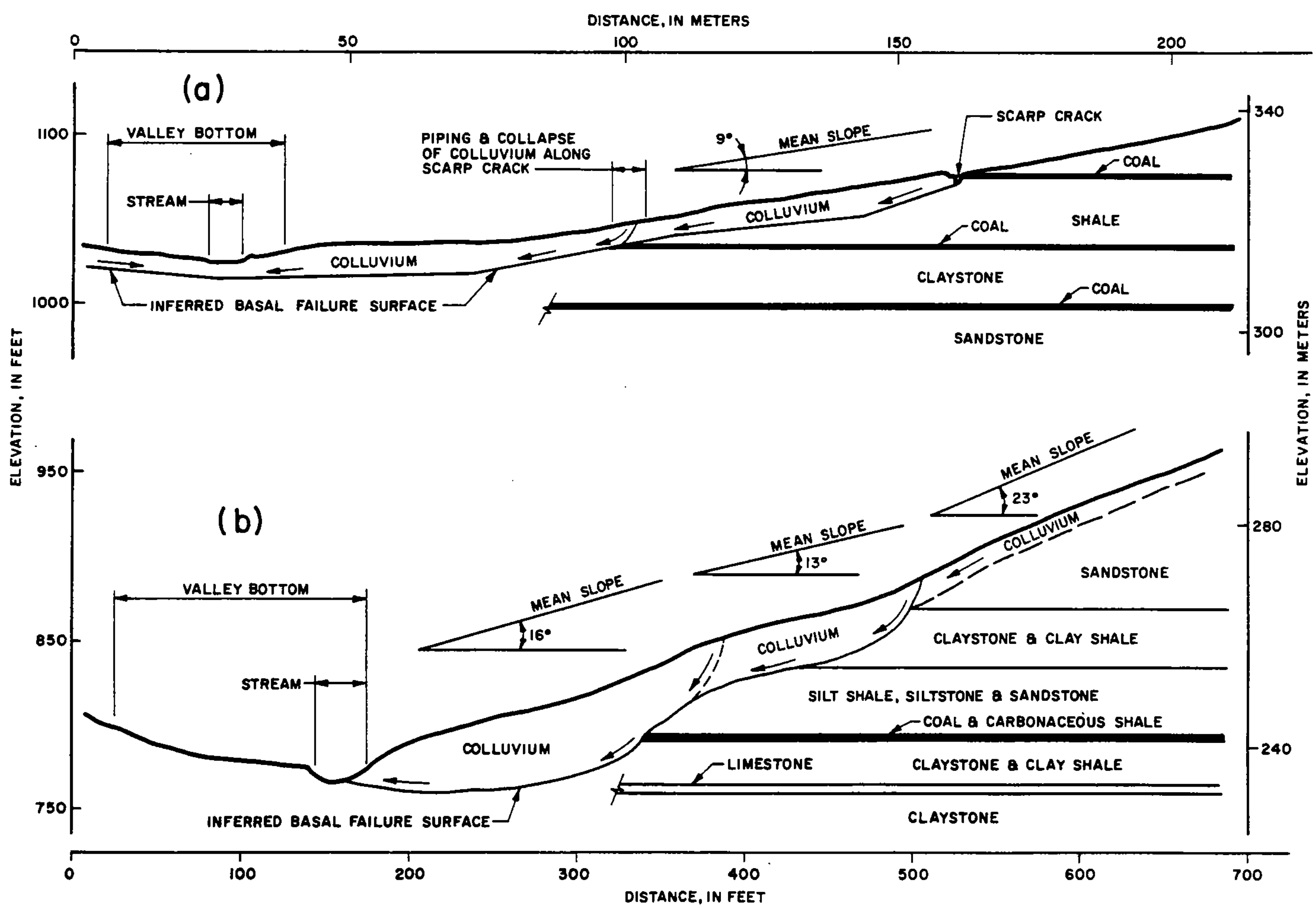

Fig. 6 Cross-sections of colluvial slopes: (a) Thick interval of weak rocks, (b) Interbedded strong and weak rocks (After Hamel, 1980)

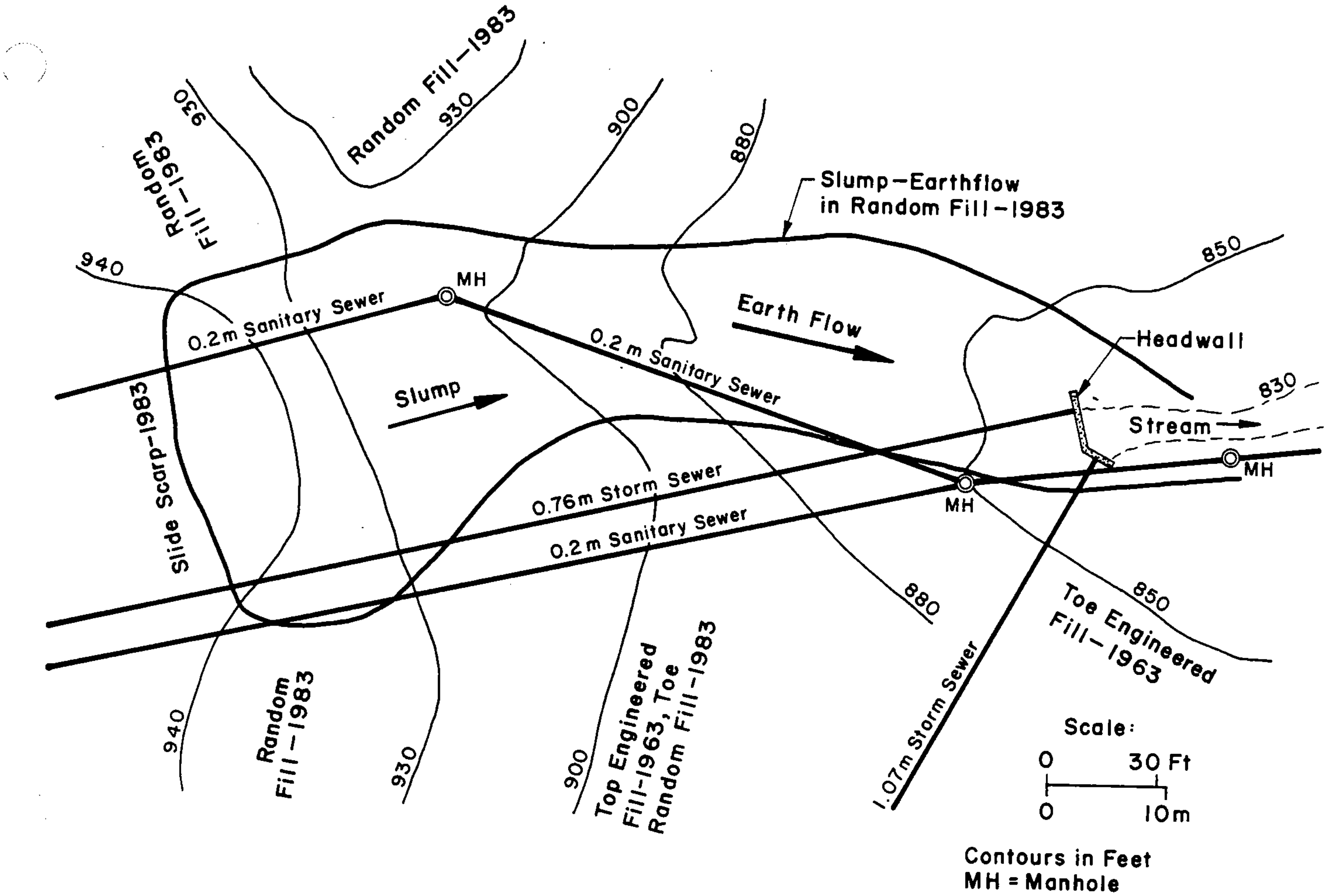

Fig. 7 Plan of typical fill slide

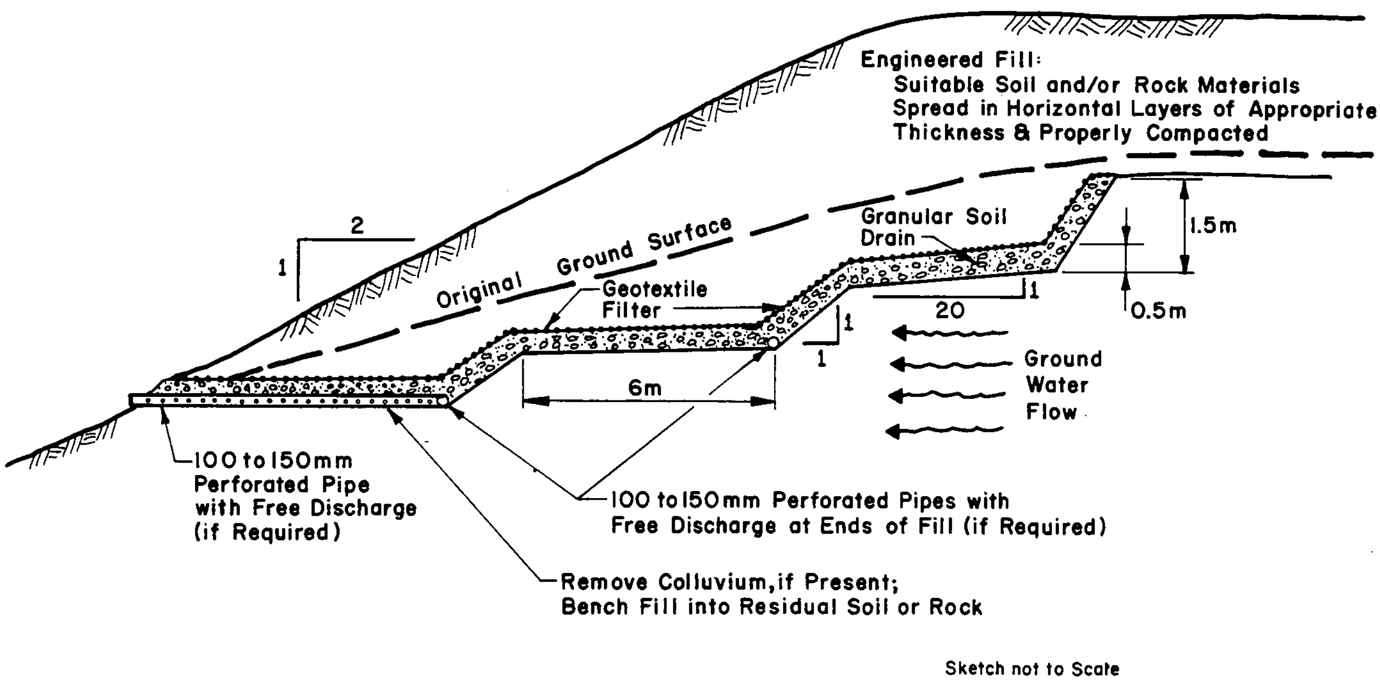

Fig. 8 Cross-section of engineered fill

CAUSES AND EFFECTS

Landslides are caused by nature's actions and man's interactions. Natural processes related to landsliding in Pennsylvania include erosion, valley stress relief (Fig. 3), weathering, and creep (Gray, Ferguson, Hamel, 1979; Hamel, 1980; Ferguson, Hamel, 1981). Precipitation is a common triggering mechanism for landslides. Abnormally heavy precipitation events, e.g., Tropical Storm Agnes of 1972, the 1977 Johnstown storm, and the 1980 East Brady storm, have caused major landsliding (Gray, Ferguson, Hamel, 1979; Pomeroy, 1980, 1982, 1984).

Man's activities leading to landsliding are commonly associated with site development and other construction. Surface excavations in rock may result in rock falls. Surface excavations in soil remove lateral support and sometimes trigger landslides; this is particularly common in colluvial slopes of the Appalachian Plateaus (Gray, Ferguson, Hamel, 1979; Hamel, 1980; Hamel, Adams, 1981). Subsurface excavations can contribute to or even trigger landslides. Slope movements induced by coal mine subsidence are not uncommon in the Appalachian Plateaus.

Construction activities can also cause landslides by loading a slope and by changing established patterns of surface and subsurface drainage. Fill placement is a common agent of slope loading as well as drainage changes, e.g., blocking of surface water flow paths and/or subsurface drainage outlets including spring discharges. Poor fill is responsible for many landslides, particularly in the Appalachian Plateaus (Fig. 7).

The effects of landsliding in Pennsylvania are numerous. General effects include distress and damage to property, structures, facilities, and utilities, (Fig. 9); traffic delays and detours (Fig. 10); continuing maintenance requirements, e.g., along transportation routes; and litigation regarding damages. Cost data on landslide damages in Pennsylvania are sparse. Fleming and Taylor (1980) presented landslide damage costs for Allegheny County (Pittsburgh and suburbs, Fig. 2) from 1970 to 1976. Estimated annual landslide damage costs ranged from $1.3 to 4.0 million over this seven year period with an average of $2.2 million per year. (The maximum annual cost of $4 million was for 1972, the year of Tropical Storm Agnes.) Unpublished data from the Pennsylvania Department of Transportation (1977) indicate that $6 million was spent on repair of landslide damage along state roads in Allegheny County for the 6½ year period from January 1971 through July 1977.

Injuries and fatalities due to landslides are fairly rare and result mainly from rock falls in highway slopes and soil falls in trench excavations (Ackenheil, 1954). A 115 m^3 rock fall from highway cut along the Ohio River northwest of Pittsburgh (Fig. 2) in December 1942 crushed a motor coach; 22 persons were killed and four were injured (Ackenheil, 1954; Gray, Ferguson, Hamel, 1979). A 230 m^3 rock fall occurred in Pittsburgh (Fig. 2) in February 1983 during remedial excavation of a highway slope with a long history of rock falls. This rock fall crushed three vehicles; two persons were killed and one was injured (Fig. 10). Every year, one or more construction workers are typically injured or killed in cave-ins of trenches and other excavations in Pennsylvania.

Fig. 9 House destroyed by landslide involving fill over colluvium, Pittsburgh suburb, May 1974 (Photo by J.V. Hamel)

INVESTIGATION AND TREATMENT

Landslides are natural geomorphic processes. For natural slopes in remote areas or in other areas where there will be no adverse effects, landslides are simply left alone. In critical areas, landslides in natural slopes are treated similarly to those in man-made slopes.

Man-made slopes include those modified by excavation and/or fill placement. Proven technology is available for minimizing the adverse effects of landslides in man-made slopes. This technology includes procedures and techniques for investigation, design, construction, and maintenance (Schuster, Krizek, 1978). The most critical item here is understanding the geotechnical framework of the slope (Hamel, 1980). If this framework is understood, it is not usually difficult to implement proper design and construction procedures, provided that the economics of such procedures can be justified.

Deep-seated rock slides are treated on an individual basis because of their rarity. Standard procedures are available for dealing with the more common shallow, slab-type rock slides, rock falls, and rock topples. In excavated slopes, the key thing is proper design and construction. Controlled blasting minimizes overbreak and rock loosening. Potentially unstable rock blocks can be removed during excavation and drop zones can be provided to catch other rock blocks. Support or stabilization measures, e.g., rock anchors, rock bolts, wire mesh, retaining structures, drainage features, can be provided (Schuster, Krizek, 1978, Chap. 9). In natural rock slopes, the options are usually limited to removal of potentially unstable rock masses or supporting and/or stabilizing such rock masses.

Fig. 10 Vehicles crushed by rock fall, Pittsburgh, February 1983 (Photo by V.W.H. Campbell, Jr., Pittsburgh Post-Gazette)

With colluvial slopes, the key thing is recognition of old landslide masses (Gray, Ferguson, Hamel, 1979; Hamel, 1980; Hamel, Adams, 1981). Colluvial masses, especially the larger ones, should be avoided to the extent practicable. If they can not be avoided, colluvial masses can sometimes be stabilized with buttress fills or retaining structures (Schuster, Krizek, 1978; Chap. 8). Stabilization of a colluvial mass by excavation alone generally requires removal of virtually the entire mass in order to ensure stability. This is seldom practical with the larger colluvial masses. Improvement of subsurface and surface drainage is an important component of stabilization measures for many colluvial slides though drainage by itself may not be sufficient for slide stabilization.

The stability of fill slopes begins with the foundation. All soil foundations must be carefully investigated. Fills placed on colluvium are seldom stable in the long term. Assuming a stable foundation, good grading practices are mandatory to ensure a stable fill slope. These practices involve engineered fill with proper materials and placement techniques, the latter including keying or benching the fill into stable underlying soil or rock and adequate compaction of the fill materials (Fig. 8). Surface and subsurface drainage must also be provided for a stable fill.

Some municipalities in Pennsylvania have grading codes and ordinances intended to ensure appropriate geological and engineering investigation, design and construction of excavated slopes and fill slopes. Many of these codes and ordinances fail to achieve their objectives because of limited or nonexistent capability for knowledgeable review and follow-up inspection and enforcement of their provisions. Rigorous review, inspection, and enforcement are essential for implementation of the state-of-the-art slope engineering and construction necessary to reduce landslide damages, particularly in the Appalachian Plateaus of western and northern Pennsylvania.

ACKNOWLEDGEMENTS

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The figures in this paper were drawn by James H. Wylie, Technical Illustrator.

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MARYLAND ROUTE 31 SINKHOLE

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ABSTRACT

This paper is a history of the sinkhole that occurred on Maryland Rte. 31 on April 31, 1994. The presentation includes a series of 35 millimeter slides and covers the following:

- The description of events surrounding the discovery of the sink.
- Description of the area geology where the sink is located.
- Discussion of geophysical studies that were made in the area after the collapse.
- Discussion of the boring survey.
- Review of the final repair.

MARYLAND ROUTE 31 SINKHOLE

During the early morning hours of April 31, 1994, a sinkhole opened in the roadway of Maryland Route 31, a two lane highway in rural central Maryland. The opening was approximately 10 M (30)' in diameter and 6-7 M (18'-22') deep. The location was just west of a gentle vertical curve in the highway so it was not easily visible to westbound motorists. An individual driving a small van was not able to take evasive action and a fatal accident was the result.

In order to quickly reopen the road to traffic, the sinkhole was immediatly refilled with large rock and repaved. The result was a roadway that required weekly maintenance in order to remain operational. Following the temporary repair, extensive boring and geophysical studies were made in order to determine if more collapse was imminent.

A more comprehensive repair was made to the roadway in the fall of the year.

EVENTS SURROUNDING THE DISCOVERY OF THE SINK

During the early morning hours of April 31, 1994, a sinkhole opened in the roadway of Maryland Route 31. The opening was approximately 10 M (30') in diameter and 6-7 M (18'-22') deep.

The location was just west of a gentle vertical curve in the highway so it was not easily visible to westbound motorists. An individual driving a small van was not able to take evasive action and a fatal accident was the result.

Maryland Route 31 is a state road connecting the towns of Westminster, New Windsor, and Libertytown. The Carroll County section between Westminster and New Windsor was built in 1966 as a relocated highway replacing an older road, still in service, now numbered MD Route 852. Rt. 31 consists of two, 3.8 M (12') lanes with 3 M (10') shoulders. In the east of the intersection of Medford Road, a county connector, the eastbound shoulder has been modified to be an acceleration lane for trucks entering the highway from a quarry entrance a little to the west. Side ditches were sod lined.

The sink and the associated accident removal caused the temporary closure of the highway. Due to the need to reopen the roadway as quickly as possible, the hole was simply filled with quarry rock and paved. Little or no removal of the roof was made. The result was a resumption of service, but weekly pavement maintenance and daily monitoring was required in order to maintain service.

A comprehensive study of the area was made and included borings, aerial infrared photography, ground penetrating radar, terrain conductivity, and natural potential readings.

Based on these studies, a contract was developed for permanent repair. The repair consisted of excavation of the sink down to rock and backfilling with rock to support a reinforced concrete layer with more rock and hot mix asphalt pavement placed over it. Side ditches in the area were paved. The repair was effected in October and November, 1994.

GEOLOGIC SETTING

The geologic setting of this highway is in a gentle valley named after a Precambrian formation known locally as the Wakefield Marble which underlies it. The valley sides are held up by meta-volcanics (phyllites). The marble is generally white, finely crystalline with few impurities. It is quarried for a number of uses including highway aggregates and, in neighboring Frederick County, the manufacture of portland cement. The strike of the contact between the two rock types in the area of the sinkhole trends to the north east. Evidence also suggests a fracture trace that parallels the contact in this area. Water tables are deep in this location presumably due to the location of an active quarry located about 600 M (2000') to the south west of the sink location. Residual soils from the marble formation are typical red clays. The residual soils from the meta volcanics are micaceous silty clays.

There are a number of contacts between the marble and the meta volcanics because the formations are intensely folded. The folds have plunging axes which result in curving outcrops in some areas.

The section of highway where the sink occurred trends east-west. It crosses several contacts between the marble and the meta-volcanics. The collapse occurred at one of these contacts. Photographs of the sink and of the excavation made during its repair show saprolite from the meta-volcanics on the west side of the opening and classic marble "pinnacles" on the east side of the opening.

The contact does not appear to be vertical and the meta-volcanic saprolite and soils may have arched out over the sink. This may explain why there was no noticeable settlement preceding the collapse.

GEOPHYSICAL STUDIES

The fact that a pavement failure resulted in a fatality was an immense shock to the Highway Administration equivalent to the collapse of a bridge. The State Highway Administrator noted that like all states, Maryland has a comprehensive bridge inspection program, but lacked an equivalent formal program for inspecting roads in areas of potential sinks. The need to develop one was obvious.

The Engineering Geology Division, a part of the Office of Materials and Research was directed to make a comprehensive study of the affected section of highway and also to develop a program for prioritizing other highways in the state where sinks might occur.

To make the local study, consultants were invited to make geophysical studies of a 450 M (1500') segment of the highway. The several surveys included Ground Penetrating Radar(GPR), Terrain Conductivity(TC), and Natural Potential(NP).

In addition, aerial infrared video photography was run on this highway between the towns of Westminster and New Windsor which includes this segment.

The section of roadway where the sink occurred lends itself to geophysics because there are no underground utilities, no metal traffic barriers, and due to the availability of other roads, traffic could be detoured during the tests when necessary.

The results of the geophysical surveys was mixed, complete analysis will be available after the location survey data of the excavation is completed.

The aerial infrared video photography produced no results that could be confirmed by the boring survey. However, it is noted that the infrared survey was done in early April when air temperatures were in the mid fifties. Perhaps better results could be obtained during very cold or very hot periods. Based on the work done, we believe that for an anomaly to be detected by infrared, it would have to be so close to the surface that adequate warning would not be realized.

The (GPR) survey showed promise and gave detailed information within its range. However, the range was limited to about 4.5 M (15'). We believe that within the depth that was surveyed, no cavities existed to be found.

Two TC surveys were run. They were the EM 31 which has an effective depth of 4.5 M (15') and the EM 34 which has an effective depth of 15 - 30 M (50'- 100').

The TC surveys both showed strong anomolies in the area of the sink which had been temporarily repaired at the time. In the study area outside the zone of the major sink, they also showed anomolies at the thicker parts of the overburden which would be the likely places for sink formation. They appear to be useful, rapid survey devices for setting up a limited boring program.

The disadvantage of the TC survey method is that it is strongly influenced by the presence of metal utilities and/or traffic barriers. Traffic on the roadway had to be stopped during these tests.

The NP survey provided the most comprehensive geophysical information. The final report consisted of profiles for each survey line and a plan view made from the combined profile information. From the highway engineer's perspective, the planview is the most useful tool for planning highway geometrics, storm water management, and boring surveys. The disadvantage was that it required survey lines that were off the highway right of way, which is not always possible.

THE FINAL REPAIR

The plans for the permanent repair consisted of:

- Excavating the soil down to rock
- Bridging the sink's outlet with large boulders
- Choking the boulders with smaller rock
- Placement of a .6 M (2') reinforced concrete layer
- Backfilling the excavation with rock fill to grade
- Replacement of the asphalt pavement
- Paving the side ditch

Filter cloth was placed on the sides of the excavation in order to prevent migration of the soils into the rock fill.

The excavation was initiated in October of 1994, 6 months after the collapse. The soil was wasted in a near by field.

The contract actually consisted of the excavation and backfilling of two sink locations. The two sinks were separated by about 650 M (800') of roadway. Replacement of the pavement and the paving of the side ditch were part of the contract.

Because there were two excavation sites on the project, the contractor elected to use two backhoes. During the deepest part of the excavation the excavators worked in tandem, much like a bucket brigade.

Material was removed from the sites to a waste area that was contiguous to the project. Since access to the waste site did not require over the road travel, both 10 wheeled dumps and off road trucks were used.

The property owner of the field where the waste was placed has plans for industrial expansion which includes additional large flat roofed buildings, parking lots, access roads and storm water management facilities. Since this field has a history of sinkholes, we wonder how successful the expansion will be.

When the contract was prepared, it was assumed that a point would be found at the bottom of the excavation that could be bridged with large boulders, affectionately called "goonies" in Maryland. As it happened, the opening at the limit of excavation was larger than any of the largest goonies that were available or that could be handled by on site equipment. We elected to make our bridge, or plug, out of concrete and accordingly M^3 (44) cubic yards of concrete were poured into the hole.

The rock backfill placed over the concrete plug met the following specifications:

STONE FOR CAVITY BACKFILL. Stone for cavity backfill shall be hard, durable, angular, resistant to weathering and water action, free of overburden, spoil, shale, slate, and organic material.

SIZE REQUIREMENTS	
2 Ft Maximum	
SIEVE SIZE	PERCENT PASSING
1 in.	0-50
No. 200	0-10

QUALITY REQUIREMENTS	
TEST AND METHOD	**SPECIFICATION LIMITS**
Apparent Specific Gravity T 85, min	2.50
Absorption T 85, % max	3.0
Sodium Sulphate Soundness 5 cycles, 2½ - 1½ in. Aggregate T 104, % loss, Max	20

As noted above, the sides of the excavation were covered with geotextile in order to reduce infiltration of soils into the rock fill.

QUANTITIES*
(Note, that quantities are for two sinkholes)

Item/Unit	Estimated	Actual
Class 1 Excavation - Cubic Meters (Cubic Yards)	11,154 (16,500	23,400 (30,000)
Rock Backfill - Metric Tons (Tons)	24,000 (26,000)	26,806 (31,500)
Concrete Sink Throat Plugs - Cubic Meters (Cubic Yards)	150 (200)	54 (72)
Geotextile Class C - Square Meters (Square Yards)	6,800 (8,000)	5,440 (6,400)
Asphalt Paved Ditch - Metric Tons (Tons)	105 (115)	280 (312)

Most of the excavation related quantities exceeded the estimates. This was due in part to not adequately addressing the slope ratios needed for a safe working environment, and for not anticipating the excavation needed for access roads into the pit.

The excavation was originally expected to go as deep as 15 M (50'). In fact it went to 22 M (70') and could have been deeper if equipment had been capable. Once the limit of travel of the excavator was reached, it was necessary to move the machine away from the pit so that it could dig a lower working platform for itself. This amounted to a great deal of excavation of sound material and expenditure of time. When the point was reached where the excavator's working platform was out of reach to the dump trucks and the other excavator, the decision had to be made to build the plug and start the backfill. Fortunately the opening was down to a manageable size.

The contract was awarded for $549,910. The final cost was $706,110. The estimated cost had been $639,980.

Since the occurrence of this sink, six other sinks have been found along the rights of way of state roads in this valley. All of them involved the Wakefield Marble Formation, and three of them were near a contact between the marble and another metamorphic.

One of these other sinks occurred near a bridge on MD Rt. 852 which was the intended detour to be used while Rt. 31 was closed for the sinkhole repair. A 1 M (two foot) diameter hole opened up in a small stream about 22 M (70') upstream of the bridge. The stream entered the hole and the stream bed under the bridge became dry except during and shortly after heavy rains. Knowing that the ground water was still flowing under the structure, we had serious concerns for the continued stability of the abutments. A boring survey confirmed that there were cavities in the rock within the influence zone of the structure. The spread footings of the abutments were immediately reinforced with pin piles.

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REMEDIATION FOR HIGHWAYS IN KARST

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ABSTRACT

A number of remediation concepts are available for highway pavements and structures located in karst terranes. Included are: the relocation of structures and ROWs out of valleys underlain by cavity-prone limestone, dolomite and/or marble; the use of piles or piers; preloading; and selective excavation and backfill, often with grouting or dental concrete. This paper addresses two concepts that the writers have used successfully at a number of locations, dynamic compaction (or dynamic destruction) and grouting.

No matter what engineering repair is contemplated, it is necessary to have a reasonable understanding of the highly erratic karstic subsurface to formulate a cost-effective remediation plan. The exploratory procedures are somewhat different than those used in customary soil mechanics investigations for highway routes and structures, although the exploratory tools themselves are conventional. The normal suite of investigatory tools include; aerial photography, satellite imagery, geologic reconnaissance, rotary wash drilling, split double- or triple-tube core barrels, monitoring of water and grout losses, and of course, experienced inspectors and drillers. Conventional geophysical tools are often of limited use as a result of the nature of the problem and resolution limitations.

The intent of dynamic compaction is to collapse the many soil voids expected above the top of rock and use the energy of the falling mass to force low-permeability soils into the throats of rock voids. In addition, any weak or thin rock cavity roofs can be collapsed.

Grouting in karst areas can take several forms; slurry (or intrusion) grouting; compaction grouting; chemical grouting; and rock surface "seal" or "closure" grouting. Grout curtains, accelerators, thickeners, primary and secondary grouting patterns, and formed grout columns, all play a role in remediating a karst site.

Grouting and dynamic compaction are often the most cost-effective karst cures available, with the obvious exception of the no-build alternative. Engineering judgement and at least a simplistic risk evaluation is necessary in any project in karst.

INTRODUCTION

Our comments are primarily directed toward the Cambro-Ordovician-aged rocks of the Appalachians. These hard, crystalline limestones, dolomites and marbles are widespread and must be given consideration in design and construction if they exist below a project site. Similar carbonates exist in the mid-continent U.S. and the discussion presented herein is applicable to these rocks, however, they may not necessarily apply to the soft, partially indurated carbonates of Florida and the Caribbean.

These Cambro-Ordovician carbonates can develop large cavities along bedding, joints and shear zones. Both bedrock and residual soils can develop voids of significant size. Our experience has been that the influence of karst terrane on foundation selection and ground water quality are too often ignored by design professionals of many disciplines, not just those involved in highway design and construction.

GEOLOGIC/HYDROGEOLOGIC CONCERNS

Certainly, doline (sinkhole) formation is one of the more spectacular effects produced by solution-prone carbonates. However, many other aspects of solutioned rock must be evaluated by the geologic engineer/engineering geologist charged with developing an appropriate support scheme for any proposed roadway, bridge, slope, or storm water drainage facility constructed in karst terrane. The very likely great irregularity of the rock surface as well as the depth and extent of weathered and fractured rock is a concern in both bridge design and roadway cut slope operations. The weathering process can produce a variety of shapes as well as erratic and large changes in the physical properties of the rock and residual soils over short horizontal or vertical distances.

Open channels in the bedrock as well as soil voids can provide a direct path for undiluted, unfiltered, contaminated highway runoff to reach drinking water reserves. These same channels can control the rate of both surface and ground water flow into and out of cuts. Storm water control must recognize both the dominance of open channel flow and the nature of the materials through which water is flowing. There is the need to protect ground water quality from unplanned detention basin and/or drainage way releases as a result of doline formation directing contaminants swiftly into ground water resources. Highway storm water channels and detention basins in carbonate rock areas are highly susceptible to sinkhole formation.

Of major concern from a construction standpoint is the formation of dolines beneath roadways, bridges, drainage structures and utilities. Sinkhole formation in Cambro-Ordovician karst terrane occurs somewhat differently than sinkholes in the soft "limerock" found, for example, in Florida and Puerto Rico. In the harder, older rocks of the north, water flow and gravity forces will enlarge soil voids formed above solutioned joints, fractures and channels in the underlying rock. Water flows are often noted along the top of the relatively impermeable rock surface conducting water-softened soils into distant, down-gradient voids. Thus, the location of a large rock cavity does not necessarily reflect the future location of a sinkhole.

The result of this soil migration is an ever-growing arch supporting the surface soils above (as shown on Figure 1). An increase in water movement caused by any construction can result in increased erosion of the near-surface soil arch of Figure 1 and facilitate a failure. Additional loading and/or the removal of a portion of the soil arch by grading can also result in the formation of a sinkhole, the surface manifestation of this soil collapse phenomenon (e.g., Figures 2A and 2B). These events can occur over time periods of concern to engineers and planners, much more rapidly than the actual solutioning of sound rock. It is generally this soil collapse which causes sinkhole problems in pavement or drainage facility areas. A cavern roof collapse such as shown on Figure 2C is a less likely occurrence in the Cambro-Ordovician carbonates discussed in this paper as a result of their high strength when intact (on the order of 500 kPa). Of course, acid rain or the introduction of low pH fluids into the subsurface can enhance dissolution of both residual soil and carbonate bedrock.

KARST CROSS-SECTION

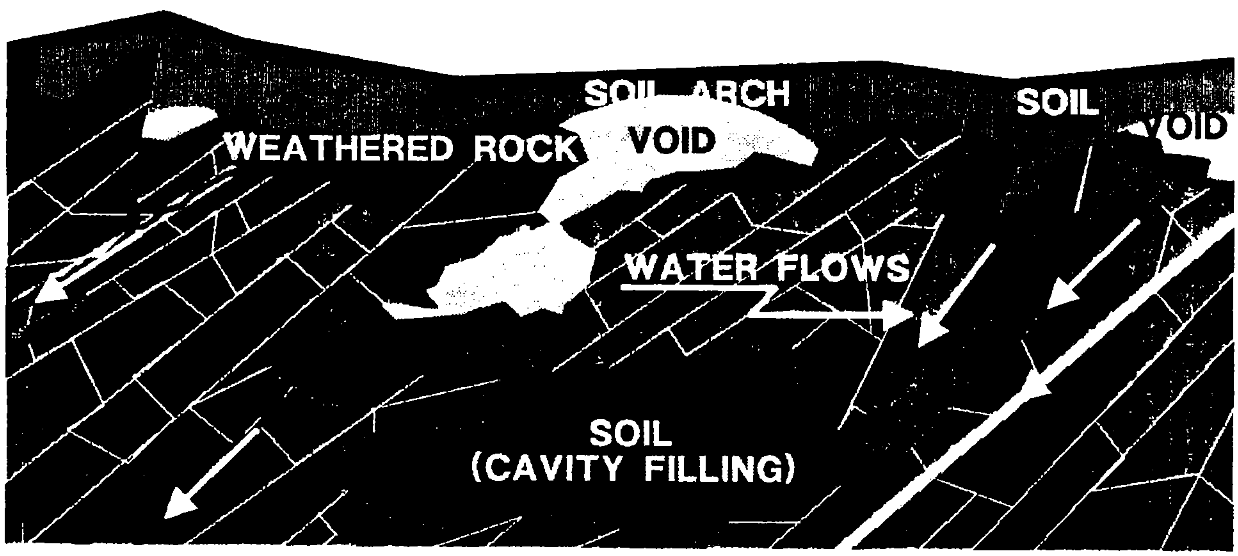

FIGURE 1

While the popular misconception of sinkholes being formed as a result of present-day solutioning is falling by the wayside, it is still hard to explain to a designer the difficulties of quantifying flow rates and direction of ground water movement in a karst regime. We are all trained to think of surface water as moving slowly downward and outward in ever widening circles with many of the contaminants being adsorbed in the soil and diluted with time, while having flow rates on the order of 100 feet per year. In solutioned carbonates, contaminant slugs can be introduced into ground water, without dilution or adsorption, virtually instantaneously and delivered 100 feet away in hours, not days. Ground water discharge in the form of springs can vary from location to location with season or precipitation events. LNAPLs (e.g., gasoline) floating in underground streams have exploded and destroyed the overlying roadway. The ground water concerns that can be posed by the existence of solutioned carbonates below highway structures or pavement is probably limited only by one's imagination.

SOLUTION SINKHOLE

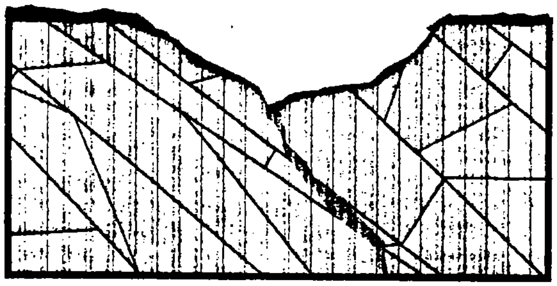

FIGURE 2A

CAVE COLLAPSE SINKHOLE

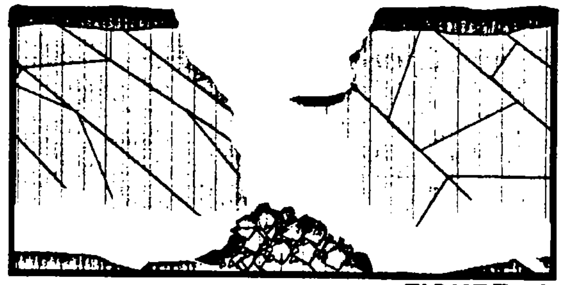

FIGURE 2B

SUBSIDENCE SINKHOLE

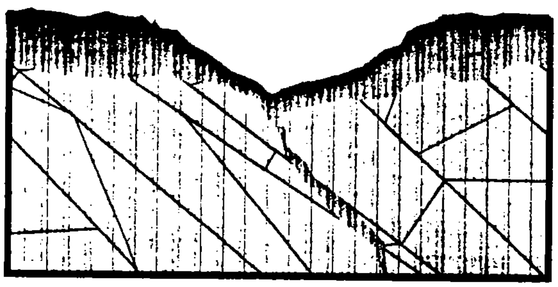

FIGURE 2C

BURIED SINKHOLE

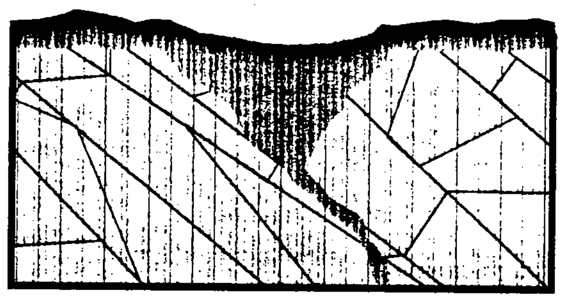

FIGURE 2D

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SUBSURFACE INVESTIGATIONS

One cannot select or perform an appropriate remedial procedure in these rocks without a reasonable understanding of the site surface and subsurface conditions. The need for remediation along a right-of-way can become evident as a result of our knowledge of the conditions that must be addressed when designing and constructing the roadway or, less pleasantly, as a result of an unforseen failure. Of course, the first step in designing a remediation program is to bring to bear the engineering geologic knowledge of the nature of karstic subsurfaces obtained from other projects. This should be coupled with a review of the available geologic data from either the original design studies (which may or may not be helpful) or from federal and state mapping (published or **unpublished**). As a result of the normally large ground water resources in carbonate rock areas, as well as the potential for aquifer contamination, geohydrologic study sources should be reviewed.

Aerial photographs taken over a period of at least a decade are invaluable. False color or infrared imagery can also be useful. The annual precipitation and drought cycles can highlight certain water related features at different times of the year, especially using the variety of imagery available today. Features visible in early photographs may be obscured by later development. Persistent lineaments and circular shapes are particularly suspicious when they are observed on a number of photographs taken over extended time periods. Anomalies noted at the intersection of fracture traces or forming a line of their own are to be regarded with great suspicion and can allow the projection of a suspect area into the as yet unrevealed area planned for construction. Photographs taken after periods of high precipitation can reveal interesting patterns of subsurface and near-surface water movement which can both aid in developing a geologic model and provide an indication of existing drainage/recharge patterns. Areas that farmers avoid during cultivation often represent rock pinnacles or persistent sinkholes. Forested areas in otherwise fully farmed lands generally indicate areas of shallow rock or possibly intense sinkhole activity. Changes in vegetation can sometimes indicate incipient sinkhole activity.

"Ground truthing" photographic features in combination with a review of the available geologic information is used as the bases for a reconnaissance of the planned remediation area and the surrounding locale.

The amount of useful information that can be developed by an experienced engineering geologist from this preliminary phase of work is surprisingly large, at least in areas where highway construction has not already altered the landscape. The findings can usually allow a number of decisions to be made as to possible route changes, the extent of the area(s) requiring remediation and the possible need for additional field investigation.

If further investigation is warranted, the authors have found that direct measurement of the subsurface conditions in at least representative areas is the best procedure (e.g., Fischer and Canace, 1989). Geophysical means (ground-penetrating radar, resistivity, conductivity, seismic reflection/refraction, etc.) are frequently used to examine the subsurface prior to roadway construction as a result of their ability to cover large areas in a relatively economical manner. However, the highly variable nature of the karst subsurface, the often clayey nature of the residual soils found above both decomposed and competent rock and resolution difficulties in defining small soil or rock voids can make the interpretation of the data from these tools extremely misleading without input from direct methods such as test borings, pits and probes.

The authors' strongly believe in the value of carefully drilled test borings using experienced drillers and inspectors who will communicate with each other. The use of rotary wash boring techniques instead of augers allows one to use drilling water losses as a diagnostic tool. The experienced driller or inspector can tell much from the sound and feel of the rig as it advances through different materials, even after the complete loss of drilling water. This ability can prove quite valuable in determining the existence of soft soils or voids above rock, the nature of cavity fillings, and/or the degree of rock weathering encountered during a core run. Pinnacled rock or open, steeply dipping cavities will often force the drill string to angle or "push-off". The direction should be noted prior to breaking the rod string. Double- or triple-tube, **split** core barrels, Nx-size or larger, should be used for sampling rock. The split barrel is the only way to view recovered core, cavity filling, clay-filled seams, weathered and/or broken rock zones in a near in situ condition. "Banging" a zero RQD core run out of a conventional, non-split core barrel gives an unacceptable picture of the downhole conditions.

Used in conjunction with appropriate test borings, both test pits and pneumatic probes can be quite useful. The test pit operations should be performed under knowledgeable technical supervision, as much can be gained from viewing the side walls and bottoms of test pits that encountered rock. Discolored residual soils and old, buried sinkholes (Figure 2D) in the pit walls are indicators of potential future problems. The variable rock surface, with or without solutioned channels, open or soil-filled seams, and/or soil voids that are often exposed in the test pit excavation, is a wonderful indicator of what one will see in future remediation or excavation programs.

Air-track probe holes, when used appropriately, can also be a useful exploration tool. A experienced air-track operator can often feel changes in the stiffness of overburden soils. Recording penetration rates can provide a measure of the changes in material properties, both within one probe hole and across a number of probe holes. Losses in air circulation are obvious indicators of cavities and often, interconnections between holes can be observed as drilling air exits from a nearby probe hole. A concern with using air-tracks in karst areas is that commonly encountered soft soils either above the rock or in rock cavities can clog the bit and\or close around the rods making it difficult or impossible to advance or retrieve the drill string. Open joints or bedding planes often swallow bits and rods. Thus, percussion probes may not be as economical as ordinarily envisioned from their use in more conventional geologic environments.

DYNAMIC COMPACTION/DYNAMIC DESTRUCTION

Dynamic compaction (DC) is usually used to densify granular soils for improved foundation performance under both static and seismic loads. It is generally quite effective in compacting loose or moderately compact, clean or silty sands. DC has even been used to compact mine spoils and miscellaneous fills (including garbage), but not necessarily in preparation for supporting structural loads. Thus, most of the available literature on the use of DC (e.g., Leonards, et al, 1980) deals with density requirements and performance prediction for granular soils. Less information is available for the use of DC in conjunction with karst (e.g., Henry, 1989) and then only in reference to Florida sands and solutioned "limerock".

The solution-prone carbonates of the Appalachians (marbles, dolomites and limestones) are much more competent when unweathered and are generally overlain by clayey, low-permeability residual soils. Thus, the use of DC in Appalachian or mid-continental U.S. karst locations is not common. The densification of granular soils is not usually the intention in karst area remediation, the destruction of soil voids and cavity roofs is the primary purpose, i.e., dynamic destruction (DD) not DC.

Of course some compaction of any loose, sandy soils (often found immediately above the rock) does occur. The depth of the hole that results when a heavy weight is dropped several times in areas of weak, clayey soils indicates that much of the energy of the intial drop is likely used to shear soils along the sides, with later drops providing more compaction to the soils below the bottom of the crater generated. When one observes the results of a DD operation, it is obvious that compaction likely occurs as a result of the confining effect of the adjacent soils below the usual 5 to 10 foot crater depth.

Vibration measurements (taken at the ground surface in these materials) seem to confirm that the energy of the falling weight is primarily converted to vertical compression and shear. Recorded motions resulting from the dropped weight are principally longitudinal and transverse, indicating that shear and Rayleigh or Love wave motions are the dominant wave trains. Overall, measured particle velocities are not large even when dropping a heavy weight from significant heights. As an example of expected ground motion levels in residual soils, the data plotted for Figure 3 was obtained in silty clays/clayey silts (Standard Penetration Test blows of 5 to 30/foot) in generally shallow, but variable depth (10 to 30 foot deep) soils. For comparison, vibration attenuation for DC sites in granular soils is also shown on Figure 3 (from Dobson and Slocombe, 1982).

Contrary to conventional wisdom, the depth of effectiveness of the DD operation may be equal to or slightly greater than that normally estimated for granular materials. The overall depth of effectiveness in fine-grained soils is probably decreased by the inability to pump water out of the soil, but increased by the reduced soil cover and the greater drop resulting from the deeper craters generated by the falling weight. For planning purposes, the writers believe that conventional data from DC operations in granular soils sites can be used to **estimate** the depth of effectiveness of DD at a clayey/silty soil site. A reasonable formula for use is that the depth of influence in meters, is proportional to $0.5\sqrt{Wh}$; where W is the mass of the falling weight in metric tons and h is the height of the drop in meters (e.g., Dobson and Slocombe, 1982).

In using DD procedures, the intention is to first collapse any existing soil arches over voids or even shallow, thin rock roofs over cavities. The brute force concepts of DD, e.g. a 15 ton weight falling 20 meters, should impose greater forces than a heavy truck bounding down the highway. In addition, plugging the throats of rock cavities with low permeability overburden or fill material, such as residual silty clays, will reduce (but not necessarily eliminate) future soil erosion into any underlying solutioned zones.

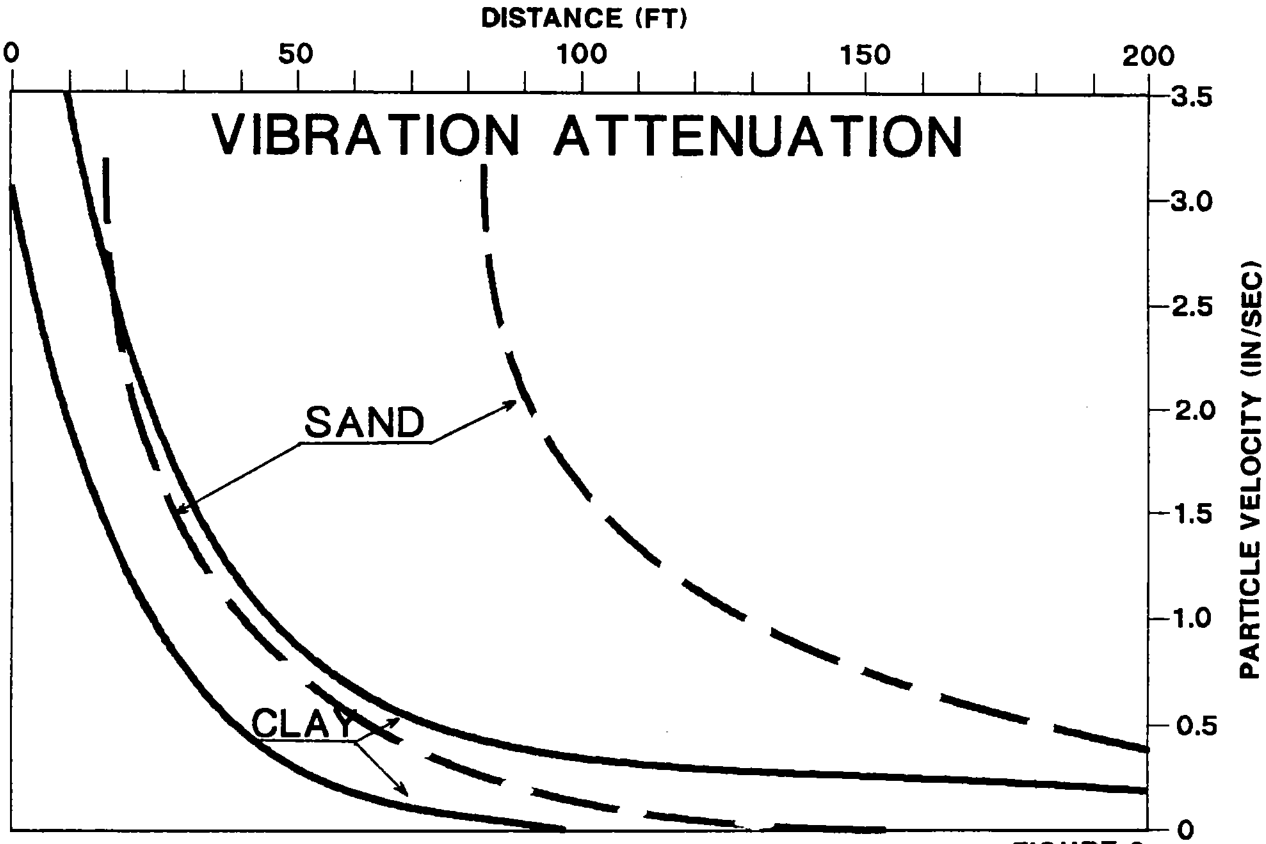

FIGURE 3

A 15 to 20 metric ton weight free-falling some 15 to 25 meters is recommended for general planning purposes. Initially, a grid pattern of 3 to 6 meters may be considered for planning purposes. However, it should be assumed that adjustments will be made in the field depending upon the nature of the areas of concern (selected by previous field investigation) and observations of actual DD field operations. One can quickly learn to tell the difference in sound that the impact of the falling weight makes when entering "squishy" ground versus an area of competent subsurface conditions.

The use of both experienced crane operators and inspection personnel cannot be over-emphasized. The field engineer/geologist should have an understanding of the expected conditions and of the many possible vagaries of karstic subsurfaces. It should be assumed that no reasonably economical subsurface exploration program will expose all soil or rock voids, nor indicate the many variations in rock and soil depths and their physical properties. The typical karst subsurface shown on Figure 1 occurs in many variations and perturbations, and these aberrations are the norm, not just anomalous. In addition, it is quite helpful to have an experienced crane operator unless one wishes to ride in the crane cab. Communication with the operator may entail the number of drops, when to stop, when to continue, when to excavate an area, and to direct (fine-tune) where the next drop should occur. Rigging and operating the crane for minimum cable damage is a very important aspect of an effective and efficient operation.

The inspection personnel should also be aware of the distances that heavy clods of earth can fly and the need for hard hats and shelter. The geotechnical inspection works better when one listens to the "sound" of the impacting weight, observes the completed "crater" and awaits a call from an experienced operator (in a bulletproof crane cab) if he spots anything unusual.

In performing the actual DD operation in the field, the writers believe that about three drops per location is a good starting point. Some areas will require additional drops, but it is rare to use a lesser number except as a form of exploration in marginal areas or at locations presumed to be satisfactory from previous exploration. Conventionally, the first drop should be about half the maximum energy of a normal drop so as not to lose the weight in a large cave.

One should expect a DD site to look like a World War I battlefield as the work progresses. Remediation of an area treated by DD can consist of; pounding crushed rock into a "crater" bottom to provide strength, placing and compacting low permeability soils, or excavating to competent rock and using concrete for backfill. The choice should be made by technical personnel knowledgeable of the expected karst subsurface variations and should be considered on a case-by-case basis.

The overall result is expected to be the need for the placement of additional fills in the remediated area after regrading and recompacting the battlefield.

TYPES OF GROUTING

One of the most widespread means of providing structural support for paved areas and drainage facilities is through remedial grouting. In general, three types of grouting are best used in carbonate rock areas. These broad categories include slurry, compaction, and "limestone" grouting. Slurry grouting essentially consists of placing a lean cement mix into the soil and/or rock voids in the area of concern. Slurry grout can be placed in stages with the use of downhole packers. Compaction grouting normally encompasses the injection of low-slump (less than 5 cm) grout to displace and/or compress the surrounding soils for greater strength. Control of the low-slump grout placement can also result in constructing small concrete columns to provide adequate structural support. The final procedure, "limestone" grouting, which in some ways combines the concepts of slurry and compaction grouting (at least in the use of both fluid grouts and grouted concrete columns), uses the available geologic understanding of the site and that gained through exploratory holes to select whether; 1) the encountered void or soft soil area is small enough to economically fill with a cement grout slurry, 2) a series of columns (using thickeners, sand or gravel pyramids, or accelerators) penetrating through the solutioned area to a sound rock surface can be installed, 3) a seal of grout placed over the entrance to the solutioned zone to prevent further soil erosion into the rock, allowing the cavity roof to support future loads, is appropriate, or 4) using additives to restrict grout take in order to control movement through and into cavities is viable. The writers' have not used chemical grouting at carbonate rock sites. Costs seem to be the main concern, but the possibility of ground water contamination must also be considered with the use of any chemicals.

We have found that the "limestone" grouting concept is appropriate for many roadways, utilities, detention basins, and lightly loaded structures and is usually the most cost-effective when used as subsequently discussed.

EXPLORATION/REMEDIATION GROUTING PROCEDURES

The intention of the exploratory portion of the grouting program is; 1) to develop subsurface information at each planned grout location, 2) to evaluate the exploratory borehole information in light of the presumed geologic model of the site, and 3) to utilize the data for the selection of a grouting program appropriate to the specific borehole location and its immediate surroundings. Each successive grout hole thus expands the grouting engineer's understanding of the subsurface and allows the appropriate grouting procedure(s) for that location to be determined.

The authors believe that a typical exploration/remediation grouting operation would be conducted with one or two truck-mounted rotary wash drilling rigs (with perhaps air-tracks in addition) and a double-tub mixer. To blend certain additives with the grout, two Moyno-type pumps are often used. Normally grout flow would be under gravity conditions, although low pressure grouting may be used if it is deemed advantageous. If any form of "pressure-grouting" (or compaction grouting) is used, care must be taken to avoid overpressurizing an area resulting in; 1) lifting the soil column above the rock, 2) hydro-fracturing of the soil, or 3) merely pushing vast amounts of grout into a gigantic cavern beyond the area of interest. Of course, for high volume projects more rigs, pumps and a large batch plant can be used.

The exploratory/grouting crew should consist of drillers and helpers who have had experience in grouting operations. The grouting specialist should have experience with karst concerns and understand the geologic model for the area to be remediated. The initial grouting procedures should be based upon the information developed from all previous geotechnical studies and exploratory drilling operations. The recommended procedures for use during initial exploratory drilling are essentially those previously summarized in the SUBSURFACE INVESTIGATIONS section, and are discussed in more detail in Fischer and Canace, 1989. After sufficient information is obtained from the initial holes, simpler rotary wash or air-track drilling can be used to expedite the project.

First, an attempt should be made to fill any soil or rock cavities encountered during drilling operations. A relatively fluid, approximately a 1:1:1 to 3:1:1 (by volume) mix of water, cement and sand should be used together with a small amount of bentonite (2% to 4%), Intraplast-N (1%) or aluminum hydroxide (<1%) to control shrinkage and enhance flowability. Generally, grout should be tremied from near the bottom of the borehole. Depending upon the drilling experience at each location, some .2 to 2 cubic meters of this fluid grout should be pumped before abandoning an attempt to fill the cavity. Judgement is necessarily involved in this criteria, and each situation should be examined on a case-by-case basis.

If the cavities are filled during the initial attempts, the grouting operation should proceed to the next hole and the procedure repeated. In general, suspect areas or known sinkhole areas should first be "ringed" with grout holes to establish a "grout wall" to avoid excess grout movement. The grout program should then proceed toward the center of the area of concern, either building strengthened columns or walls, or filling the voids. Check holes can be drilled and grout-take monitored at intermediate locations between the primary holes to evaluate the efficacy of the program. If several of the secondary holes show little to no grout-take over the borehole volume (say no greater than 10%) the primary program can be considered successful. If necessary, other tertiary holes can be used to assure the suitability of the supporting or sealing program.

If, however, excessive quantities of the fluid grout are necessary to fill the encountered rock or soil cavities, thickeners (e.g., bentonite, polymers, fly ash or baroid clays) can be added to limit grout movement only to the area of interest. Alternately, accelerators (e.g., sodium silicate, calcium chloride or other environmentally safe additives) can also be used to limit the distance that the cement grout will flow. Dual Moyno-type pumps with separate pumping hoses are generally used to supply grout and sodium silicate accelerators to the borehole entrance or, if necessary, downhole where they are mixed to shorten set times. Set times (i.e. 15 seconds to 10 minutes or more) can be controlled and tested prior to placement. The grouting program should now be directed toward filling the throat of the cavity and the soil void above the rock. The extent of total cavity filling versus using "supporting columns" of grout will depend upon a knowledge of total cavity size, roof thickness and imposed loads.

Entrance holes to large cavities within the rock can often be filled by pouring sand and/or gravel down an exposed cavity, forming a pyramid of granular fill within the cavity and then grouting the pyramid into the throat with a thick mix. Once the throat is "choked", slurry grouting procedures could be used to fill the soil cavity which likely exists above and near the rock cavity entrance.

DISCUSSIONS

In developing any remediation plan, one must always be aware of the geologic model for the area of concern. How much do we know about the rock surface? Are the rocks flat-lying? Was rock recovery over-estimated as a result of the use of the wrong core barrel? Were the solutioned zones soil-filled or actual voids? Were drilling water losses recorded correctly (or was an auger used)? How much background information is available? Does aerial imagery show significant lineaments, etc? Where is the water table? Is ground water used domestically in the area? Without a reasonable understanding of the subsurface conditions, selecting an appropriate remediation scheme is a shot in the dark and may cause as much harm as good.

The authors have used the DD and grouting alternatives discussed herein on a number of carbonate rock projects and have found them to be both reasonably economical and effective. However, they are not the only available solutions (e.g., Fischer and Canace, 1989).

For heavy structures, the more traditional support schemes such as piers, pile or caissons placed on top of the rock may be appropriate at deep rock sites. However, the authors would tend to spread the load over a larger area keeping it well above possible large cavities rather than concentrating it atop solution-prone rocks. It is almost always prudent to have at least one well-monitored probe hole at each foundation location to reduce the possibility of concentrating the full weight of a major foundation element directly over a rock cavity. In considering piles, neither wood nor H-piles are normally feasible as a result of their brittleness (wood) or flexibility (H-piles) when driven to an erratic, generally sloping rock surface.

In selecting a remediation scheme, on must also understand the possible effects of disturbing the historic ground water flow patterns with either the planned construction or the planned remediation program. Ground water, whether it is under vadose, confined or unconfined conditions is of the greatest concern in pavement design and performance, even under the best of subsurface conditions. It may also be most important in selecting an appropriate remediation program. It does little good to close a sinkhole and merely divert the water flow to another location, eventually resulting in another sinkhole at a new location. In addition, both grouting and DD can be less effective and/or costly in areas with a high water table. There should also be concern for the quality of the surface water when deliberately diverted toward or into karst features or toward areas underlain by sinkhole-prone formations.

In any remediation program in karst, we will rarely have enough information to be as positive about the success of the work as we would at sites with a more uniform subsurface without an inordinate investment in manpower and money. The geotechnical engineer/engineering geologist who plans and implements a remediation program must make the interested parties aware of the level of risk and the use of judgement in his work. It is possible that sites will have to be revisited after a period of time to complete the DD or grouting operation. We cannot over-emphasize the need for knowledgeable construction and post-construction inspection. Karst is a bitch.

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Management of the Discharge and Quality of Highway Stormwater Runoff in Karst Areas
A Progress Report

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ABSTRACT

Stormwater runoff from highways is a documented type of nonpoint-source pollution of surface and groundwater. Local hydrogeologic conditions influence the impact of highway runoff on groundwater quality. In particular, highway runoff may have a deleterious effect on groundwater quality in karst terranes, where soils may be thin or nonexistent and where groundwater recharge occurs directly through fractures, sinkholes, and sinking streams.

P.E. LaMoreaux & Associates, Inc. (PELA) and the University of Tennessee (UT) Institute for Geotechnology are conducting a study of potential impacts to groundwater quality resulting from highway stormwater runoff in karst areas for the Federal Highway Administration (FHWA). The primary objective of this investigation is the development of a method for improving the quality of highway runoff draining into karst aquifers through sinkholes. A secondary objective is the identification of sensitive karst areas in the United States. Most studies of highway runoff and groundwater quality have been conducted in areas with relatively thick soils, which immobilize many runoff pollutants. Results from such studies may understate the impacts of highways in karst areas.

Literature has been reviewed regarding the quality of highway runoff, its potential impact on karst groundwater, and stormwater runoff treatment technology. Highway runoff transports solids, heavy metals, nutrients, bacteria, deicing salts, herbicides, and hydrocarbons from highway surfaces and rights-of-way. Impacts to karst aquifers by these highway-related pollutants have been documented. Other impacts associated with highway drainage in karst areas include diversion of water away from aquifer recharge features, induced subsidence and sinkhole collapse, and flooding and sedimentation of sinkholes. Stormwater runoff treatment technologies reported in the literature involve diversion, sedimentation, infiltration, filtration, and adsorption through the use of vegetation, swales, berms, gabions, and retention/detention ponds. Peat filters are also being used to treat highway runoff entering sinkholes.

In this study, laboratory testing of stormwater treatment methodology is being conducted to aid in the design of a runoff treatment system. Field evaluations of the system will be conducted at two locations, including a site in Knoxville, Tennessee. More than 125 potential sites for field testing have been visited and evaluated in each of fifteen states participating in this study: Arkansas, Florida, Illinois, Indiana, Kentucky, Maryland, Minnesota, Missouri, New York, Oregon, Pennsylvania, Tennessee, Texas, Virginia, and Wisconsin. In addition, maps have been prepared to delineate areas where highways may pose threats to karst groundwater quality or where highways may be impacted by karst geohazards, including flooding and catastrophic sinkhole collapse.

Whenever possible, highway alignment and other engineering controls should be designed to minimize the runoff of highway stormwater drainage into sensitive karst aquifers. Runoff treatment methods should be used only in areas where there is no feasible alternative to subsurface disposal of stormwater runoff.

INTRODUCTION

P.E. LaMoreaux & Associates, Inc. (PELA) is conducting an assessment of potential groundwater contamination by highway stormwater runoff in karst areas under contract to the Federal Highway Administration (FHWA). The primary goal of this investigation is the development and field testing of practical remedial measures for treating highway runoff in karst settings. Laboratory studies of potential runoff treatment methodology are being conducted under a subcontract with the Department of Civil Engineering at the University of Tennessee (UT). A secondary objective of this project is the identification of sensitive karst areas in the United States (U.S.).

Although previous studies have addressed the impacts of stormwater and highway runoff on surface water, relatively little attention has been directed toward assessing its impact on groundwater, especially in karst areas. Dissolution of bedrock (usually limestone or dolomite) in karst areas results in a terrane characterized by sinkholes, sinking streams, underground (cave) streams, and springs. Groundwater in these settings is more susceptible to contamination because surface water may pass directly into the subsurface with little or no filtration by soil. Because karst groundwater typically flows through relatively large fractures and conduits within the bedrock, it may transport contamination rapidly from points of recharge (such as sinkholes) to distant cave streams, water wells, springs, and surface streams.

This paper discusses work completed during preliminary phases of this investigation and plans for future project activities. Literature is reviewed regarding the quality of highway runoff, its potential impacts on karst groundwater, and stormwater runoff treatment technology. Potential sites for field testing of a runoff treatment system have been visited and evaluated in each state participating in this study: Arkansas, Florida, Illinois, Indiana, Kentucky, Maryland, Minnesota, Missouri, New York, Oregon, Pennsylvania, Tennessee, Texas, Virginia, and Wisconsin. Areas have been delineated where highways may pose threats to karst groundwater quality or where highways may be impacted by karst processes, such as flooding and catastrophic collapse. Preliminary conclusions are presented based on the literature review and site evaluations. Finally, a plan is presented for conducting future phases of the investigation, including laboratory testing of stormwater treatment methodologies and evaluation of the runoff treatment system in the field.

LITERATURE REVIEW

The following review addresses literature regarding the quality of highway stormwater runoff, its potential impacts on groundwater in karst areas, and methods for its treatment.

Highway Stormwater Runoff

Regulatory agencies are increasingly focusing on pollution of surface water and groundwater resources by nonpoint sources. The Clean Water Act (CWA) of 1977 and the National Pollution Discharge Elimination System (NPDES) require mitigation of impacts to water quality resulting from construction, maintenance, and operation of highways and associated facilities (Yu, 1993). Moreover, the CWA requires federal agencies to cooperate with state and local agencies in developing comprehensive solutions for the prevention, reduction, and elimination of pollution (Smith and Lord, 1990).

Investigations of the quality of highway runoff and management practices for its treatment have been conducted by the FHWA, including Gupta and others (1981); Kobriger and others (1981, 1984); and Lord (1987). Ongoing FHWA research has four major objectives: (1) identification and quantification of highway runoff constituents, (2) identification of the sources and migration pathways of the pollutants, (3) evaluation of the effects of contaminants on water quality, and (4) development of runoff management practices (Lord, 1987). Other investigations of highway runoff have been performed by the U.S. Environmental Protection Agency (U.S. EPA) (Pitt, 1979; Pitt and Amy, 1973; Sartor and Boyd, 1972; Shaheen, 1975; U.S. EPA, 1977) and the U.S. Geological Survey (USGS) (Beaven and McPherson, 1978; Hardee and others, 1978; Reed, 1978, 1980; Vice and others, 1969).

Highway runoff is an important contributor of pollution to surface water and groundwater (Gupta and others, 1981). While urban stormwater runoff has been the subject of much research, serious study of highway runoff as an independent source of contamination has been conducted only within the last few decades (e.g., Sartor and Boyd, 1972; Pitt and Amy, 1973). Highway runoff has been documented as a potential source of pollution by numerous investigators, including Gupta and others (1981), Kobriger and others (1984), Novotny and Olem (1994), Sartor and others (1974), and Shaheen (1975).

Reviews of the extensive literature on highway runoff are included in Barrett and others (1993), Burch and others (1985), Dupuis and others (1985), Gupta and others (1981), Horner and others (1977); Kobriger and others (1981, 1984), Lorant (1992), and Yu (1993). Novotny and Olem (1994) and Wanielista and Yousef (1993) present overviews of stormwater management, including highway runoff quality. Hamilton and others (1984) and Hamilton and Harrison (1987) present the proceedings of international symposia on highway-related pollution.

It has been shown that stormwater runoff transports suspended and dissolved solids, heavy metals, nutrients, bacteria, road salt, herbicides, and hydrocarbons from highways. In a study of traffic-related pollution in Washington, D.C., Shaheen (1975) estimates that traffic produces approximately 0.7 g of solids per axle-km of roadway (0.0025 pounds per axle-mi). These materials collect on highways as a result of normal operation and maintenance activities. The predominant contaminants in urban runoff are copper, lead, zinc, chromium, cadmium, nickel, arsenic, cyanide, and asbestos (U.S. EPA, 1983). Pitt and Amy (1973) note that strontium, titanium, and zircon are also among the most significant pollutants in road surface runoff. Pitt and Amy (1973) and Shaheen (1975) suggest that road runoff has a greater impact *per capita* than sewage for total suspended solids and some metals (e.g., lead, zinc, copper, nickel, and chromium). The fate and transport of heavy metals in highway runoff have been studied by Bourcier and Hinden (1979), Harper (1985), and Harrison and Lonen (1977). Concentrations and loadings of highway runoff constituents have been reported in various studies (Asplund and others, 1980; Barrett and others, 1993; Burch and others, 1985; Driscoll and others, 1990; Grottker, 1987; Gupta and others, 1981; Malina and Barrett, 1994; Moe and others, 1978; Smith and Lord, 1990).

Sources of pollutants in road and highway runoff have been investigated by numerous researchers. Hydrocarbon contamination in highway runoff results from the incomplete combustion of gasoline, oil dripping onto road surfaces and parking areas, and disposal of waste oil at storm drains (Asplund and others, 1980; Latimer and others, 1990). Gupta and others (1981) note that pavement wear and right-of-way erosion contribute to highway runoff pollution, as do highway deicing and right-of-way maintenance activities. Morre (1976) discusses the environmental safety of roadside herbicide use. In a Washington, D.C., area study, Shaheen (1975) observes that street runoff contains inorganic contaminants (e.g., phosphorus, nitrogen, and chloride) derived from the local soil and vegetation, as well as from the roadway surface itself. Nitrogen and phosphorus may be derived directly from automobiles (Driscoll and others, 1990; Yousef and others, 1991). Pollution is also derived from exhaust emissions, fluid leaks, and the normal wear of tires and other auto parts (Gupta and others, 1981; Hedley and Lockley, 1975). Investigations of the products of tire wear have been conducted by Christensen and Guinn (1979) and Dannis (1974). In one study, it is reported that 0.125 g of pollutants are contributed per vehicle per km (0.0004 pounds per vehicle per mi) from tire wear (U.S. EPA, 1977). Reports have addressed asbestos contamination associated with brake and clutch wear (Jacko and DuCharme, 1973; Jacko and others, 1973). Highway-related pollution also results from highway deposition of solid contaminants acquired by automobiles in parking lots, urban areas, construction sites, farms, and unpaved roads (Asplund and others, 1982). According to Shaheen (1975), less than 5 percent (by weight) of traffic-related deposits originate directly from automobiles. However, he notes that vehicle-derived pollutants are likely the most significant because of their potential toxicity.

A significant portion of highway pollution results from wet and dry atmospheric deposition—i.e., rainfall and dustfall (Gupta and others, 1981; Irwin and Losey, 1979; Moe and others, 1982). Novotny and Olem (1994) note that rainfall in Ontario contributes significant amounts of nitrogen, copper, and nickel. Gjessing and others (1984a and 1984b) report on heavy metals (lead, cadmium, and zinc), polyaromatic hydrocarbons (PAHs), polychlorinated biphenyls (PCBs), and other chlorinated contaminants in highway runoff in Norway. They conclude that atmospheric dust transport is a major contributor of these contaminants.

Association of contaminants with dust-size particulate matter has been studied by Harrison and Wilson (1983, 1985), Hewitt and Rashed (1992), and Hvitved-Jacobsen and others (1984). Harrison and Wilson (1985) report that about 50 percent of the metals in highway runoff are associated with dust, even though it constitutes only 6 percent of the total weight of the solid materials. Pitt and Amy (1973) report that less than 10 percent of the available metals in road surface runoff occur in a dissolved form. Similarly, Hewitt and Rashed (1992) note that the particulate phase contains more than 90 percent of the inorganic lead, approximately 70 percent of the copper, and approximately 56 percent of the cadmium. They find that particulate-phase metals concentrations generally correlate with those of suspended sediments. Association of highway-related contaminants with suspended solids has also been reported by Bourcier and Sharma (1980), Chui and others (1981), Gupta and others (1981), and Zawlocki and others (1980).

Some PAHs in stormwater runoff appear to be correlated with rainfall intensity rather than suspended solids concentrations. The concentration of aliphatic hydrocarbons in runoff decreases with each consecutive flush from high intensity rainfall (Asplund and others 1980), suggesting that their solubility limits their association with suspended solids. Data presented by Zawlocki and others (1980) also indicate that most hydrocarbon compounds remain in solution in highway runoff.

The effects of precipitation characteristics on the quality of highway runoff have been reported by Driscoll and others (1990), Harrison and Wilson (1985), Hewitt and Rashed (1992), and Hoffman and others (1985). Balades and others (1983) conclude that substantial contaminant loads may result from an individual precipitation event. They state that a few short-duration rainfall events may introduce 30 percent of the annual highway-related contaminant load.

Data from numerous studies (e.g., Wada and Miura, 1984) have suggested that the majority of pollutants are discharged during the beginning of a storm and that the pollutant load decreases with time. This is known as the "first-flush" phenomenon. In particular, accumulated loads of small particle size are flushed rapidly from a highway watershed. However, when rainfall events occur close together with few dry days for pollutant accumulation, the highway surface may be clean enough to minimize the first-flush effect (Gupta and others, 1981). Hewitt and Rashed (1992) report a highly significant correlation between the length of the antecedent dry periods and the quantity of dissolved lead and copper removed from highway surfaces by runoff events.

Other factors affecting the pollution content of highway runoff include wind and traffic-related air turbulence (Asplund and others, 1980; Aye, 1979; Kerri and others, 1985), scrubbing by tires (Asplund and others, 1982), and removal by splashing onto vehicles (Chui and others, 1981). The influence of traffic characteristics, such as speed, braking, volume, and density, have been noted by several researchers (Asplund and others, 1980; Bourcier and others, 1980; Chui and others, 1981; Dorman and others, 1988; Driscoll and others, 1990; Joumard, 1987; Kerri and others, 1985). Smith and Lord (1990) note that impact on water quality is minimal from highways with average daily traffic (ADT) of less than 30,000 vehicles per day. However, several researchers find that surrounding land use has a more significant influence on runoff quality than does the amount of traffic (Gupta and others, 1981; Mar and others, 1982; Stotz, 1987). Hamilton and others (1987) note that only 5 to 30 percent of the lead, copper, cadmium, and zinc deposited on a roadway in England is removed by runoff. The remainder of these constituents may be removed by resuspension and street cleaning. Similar results are reported by Hewitt and Rashed (1992), who find that approximately 8 percent of the lead, 5 percent of the organic lead, and 3 percent of the PAHs released by vehicles are removed by highway runoff.

The effects of pavement type and drainage mechanisms on runoff quality have been investigated (Saylak and others, 1980; Stotz, 1987). Sartor and Boyd (1972) show that streets paved with asphalt have contaminant loadings approximately 80 percent higher than concrete streets and that streets in "fair to poor" condition have total solids loadings 2.5 times greater than those for streets in "good to excellent" condition. Lord (1987) reports negligible constituent loads in unpaved areas at curb-and-gutter sites, while these areas contribute about 17 percent of the contaminant load at flush-shoulder sites. Gupta and others (1981) note that the following factors affect the quality of highway runoff: maintenance policies; percentage of impervious areas; automobile age and maintenance; litter and emission regulations; use of fuel additives; vegetation type on rights-of-way; and accidental spills. Kramme and others (1985) discuss the impacts of highway maintenance activities on water quality.

Seasonal effects on runoff quality result from the use of studded tires in winter (Andersen and Bertelsen, 1991; Bourcier and others, 1980) and chlorides associated with highway deicing activities (Field and O'Shea, 1992; Kobriger and others, 1984; Struzeski, 1971). Studies of these pollutants generally address impacts to surface-water bodies and aquatic ecosystems. Field and O'Shea (1992) summarize the results of several studies of highway deicing impacts and estimate the annual national cost of salt-laden runoff at $5.4 billion.

Highway construction effects on surface waters have been noted in the literature (Chisholm and Downs, 1978; Duck, 1985; German, 1983; Tan and Thirumurthi, 1978). Embler and Fletcher (1981) report on turbidity and suspended solids upstream and downstream of a construction site in Columbia, South Carolina. Yew and Makowski (1989) address an area along the Tennessee-North Carolina border where highway construction exposed a pyritic shale, generating acidic runoff toxic to fish in nearby streams. Besha and others (1983) note little evidence of construction-related impacts on water quality at a site near Richmondville, New York. Reports focusing on methods for preventing adverse environmental impacts at highway construction sites include Gordon (1975), Horner and others (1990), and Schueler and Lugbill (1990). Cramer and Hopkins (1982), Hall and Naik (1989), and Thrasher (1983) report on the impact of highway construction activities in wetland areas. Kobriger and others (1983) address highway construction, design, and maintenance considerations for wetlands preservation.

Numerous studies have been conducted on the effects of highway runoff on aquatic environments (Dupuis and others, 1985; Gjessing and others, 1984b; Portele and others, 1982; Schiffer, 1988; Van Hassel and others, 1980). In their assessment of highway impacts on aquatic ecosystems, Horner and Mar (1985) characterize storm runoff loading by land use. Pitt and Amy (1973) find copper, cadmium, lead, and zinc sufficiently soluble to cause toxic effects in aquatic organisms. In a study of interstate runoff in Rhode Island, Hoffman and others (1985) suggest that highway runoff contributes more than 50 percent of the annual pollutant loads of solids, PAHs, lead, and zinc to an adjacent river. Little and others (1983) note that suspended solids affect water quality by reducing visibility and light penetration. In addition to diminishing aesthetic and recreational values, fisheries potential is reduced by diminished algal productivity and smothering of bottom life. Moreover, water treatment costs increase, and other chemicals, such as nutrients and metals, may be associated with particulate material. In an investigation of highway bridge runoff, Yousef and others (1982, 1984) report that heavy metals tend to accumulate in bottom sediments, floodplains, and adjacent soils.

Impacts of Highway Stormwater Runoff on Groundwater Quality

Barrett and others (1993) note that highway runoff can significantly alter the hydrogeologic environment, including groundwater quality. Parizek (1971) reports that hydrogeologic effects of highways include the "beheading" of aquifers; creation of groundwater discharge zones where road cuts extend below the water table; changes in groundwater basins and divides; obstruction of groundwater flow by abutments, retaining walls, and sheet pilings; and changes in runoff and recharge characteristics.

In an investigation of the fate of nutrients (nitrogen and phosphorus) in highway runoff detention/retention ponds in Florida, Hvitved-Jacobsen and others (1984) find that approximately 85 to 90 percent of the nitrogen and 99 percent of the phosphorus accumulates in pond sediments. Yousef and others (1984, 1986a, 1986b) have also conducted studies of the fate of heavy metals and nutrients in highway runoff retention/detention ponds in Florida. They find that most of the heavy metals are immobilized by physical, chemical, and biological processes, accumulating in the upper 5 to 6.8 cm (2 to 2.7 in) of pond sediments under aerobic conditions. The release of metals into groundwater is apparently promoted by decreased pH resulting from sediment accumulation on the bottom of the pond. Yousef and Yu (1992) further investigate the potential for soluble metals fractions percolating through pond sediments and contaminating groundwater. They find that metals tend to migrate very slowly through the sediments and concentrate in the upper 15 to 20 cm (6 to 8 in). They conclude that removal of pond bottom sediments every 25 years should minimize groundwater contamination potential. In addition, water quality was compared among three wells—one near the pond, one near a swale receiving highway runoff, and one control well. Iron, lead, and chromium concentrations beneath the swale were twice as high as those beneath the pond and up to 10 times higher than those at the control well.

The most significant metals in stormwater runoff with respect to groundwater contamination are aluminum, arsenic, cadmium, chromium, copper, iron, lead, mercury, nickel, and zinc (Pitt and others, 1994). With the exception of zinc, most are associated primarily with particulate materials and can be removed by sedimentation or filtration. Studies of recharge basins with large loads of metals have shown that most heavy metals are removed in the basin sediment or in the vadose zone. Particulate metals are filtered out at the soil surface, and dissolved metals are removed by adsorption onto the near-surface particles in the vadose zone during infiltration. Soils rich in silt and clay are generally more effective in contaminant reduction. Studies of recharge basins have shown that lead, zinc, cadmium, and copper may accumulate at the soil surface with little downward movement over many years. The order of attenuation in the vadose zone from infiltrating stormwater is zinc (most mobile) > lead > cadmium > manganese > copper > iron > chromium > nickel > aluminum (least mobile).

Schiffer (1988) discusses zinc concentrations in a surficial aquifer near a Florida highway runoff detention pond and cypress wetland during dry (April) and wet (October) conditions. Zinc concentrations in groundwater were highest near the highway, where they were as high as 220 µg/L. Concentrations farther away were generally less than 50 µg/L and never above 100 µg/L. Dissolved Kjeldahl nitrogen was twice as high in wells near the highway.

Schiffer (1989a, 1989b) discusses the effects of an infiltration basin, a detention pond/wetland system, two swales, and an exfiltration pipe on groundwater in a surficial aquifer. Metals and ion concentrations are reduced by each of the structures, but they are less effective for nutrient removal. During their monitoring, nitrate/nitrite and phosphorus levels were highest near the swales and the exfiltration pipe. Kjeldahl nitrogen, turbidity, and color values were highest near the ponds. Specific conductivity and pH were lowest near the swales, including pH values below 6.5 (drinking water standard). Lead, chromium, and copper concentrations were less than the 1-mg/L detection level at each location. Potassium and sulfate concentrations were highest near the exfiltration pipe, although they did not exceed drinking water standards. The 10-mg/L nitrate standard was exceeded in only one sample near the exfiltration pipe. It is also reported that organic compounds retained by pond sediments may impact groundwater quality. Wanielista and others (1988) evaluate the water quality improvement resulting from the use of a stormwater detention pond associated with a groundwater recharge well, and Schiner and German (1983) report on the effects of drainage well recharge on groundwater quality at a Florida site.

The effects of highway runoff on groundwater depend on local hydrogeologic conditions, including sorption processes of the aquifer material and groundwater velocity. In an evaluation of a recharge basin receiving stormwater runoff from a major highway in New York, Ku and Simmons (1986) find no significant adverse effects on groundwater quality. Similarly, Yousef and others (1986b) find limited effects at a Florida site where groundwater migrates only 10 m (33 ft) per year. Pollutants may also be immobilized at the ground surface and within the vadose zone. For example, Bell and Wanielista (1979) report low concentrations of heavy metals in groundwater, suggesting that these contaminants are retained by the soil. In areas with thick soil, natural processes may attenuate contamination in highway runoff before it recharges the aquifer. Laxen and Harrison (1977) report that lead dispersed to roadside soils is effectively captured within the top 10 cm (4 in) of soil. They suggest that the water quality impact of lead is confined to surface runoff. Bell and Wanielista (1979) report higher concentrations of lead, zinc, copper, chromium, nickel, and cadmium in surface soils near highways in east-central Florida than in subsurface soils. This was especially true for lead, which was less mobile than the other metals studied during this investigation. Results of a study by Kobriger and others (1984) show higher sodium, chloride, and metals concentrations in percolating soil water near the highway. Metals and sodium concentrations were generally higher in surface soil than in subsurface layers, but the contaminant attenuation effectiveness of the filtration process varies with soil type.

Waller and others (1984) report on an investigation of highway stormwater runoff from a busy, two-lane road in a limestone area of Dade County, Florida. They find that some contaminants (lead, zinc, manganese, and nitrogen) are attenuated in surface soils and the unsaturated zone within the limestone, apparently limiting transport of contaminants to the water table. For example, concentrations of lead and zinc were nearly 20 times lower at depth than within the upper 2.5 cm (1 in) of soil, where they were 610 and 91 mg/g, respectively. The highest metals concentrations occurred during the first in a series of storms, suggesting correlation with the length of the antecedent dry period. Results from a control site reveal low metals concentrations in soil and rock samples and little concentration variation with depth.

Because of the near-surface immobilization of pollutants, highway runoff may pose a greater threat to soil water than groundwater in many areas. Howie and Waller (1986) report high concentrations of lead (1,000 to 6,600 µg/kg), iron (490 to 2,400 µg/kg), and zinc (90 to 1,800 µg/kg) in the uppermost 15 cm (6 in) of soil under highway swales. Concentrations were much lower at a control site. Even with such high concentrations in the soil, there was no apparent groundwater impact.

McKenzie and Irwin (1988) examine the effectiveness of exfiltration trenches and grass swales for improving the quality of parking lot and roadway runoff recharging a surficial aquifer in southern Florida. In the exfiltration trenches, runoff is collected in a open trench and drained through a perforated pipe to a subsurface reservoir full of coarse aggregate. The reservoir lies partially below the water table. Degradation of groundwater quality associated with the exfiltration trenches was not detected, and lead and zinc levels were reduced. Groundwater near the swales showed evidence of biological cycling and anaerobic conditions, which may have resulted from poor drainage and highly organic soils. Groundwater concentrations of ammonia nitrogen, iron, and dissolved solids were significantly greater, and sulfate levels were significantly less near the swales than near the exfiltration trenches.

Gupta and others (1981) find substantial concentrations of fecal coliform and fecal streptococci at all of their study sites, with ratios indicative of nonhuman sources. Pitt and others (1994) report high bacteria counts in samples of runoff from sidewalks and roads. Bacteria may be removed by straining at the soil surface and through intergrain contacts, sedimentation, sorption by soil particles, and inactivation. Enteric bacteria generally survive between two and three months in soil, although survival times of five years have been documented. Viruses have also been detected in groundwater where stormwater recharge basins are above shallow aquifers.

Because soil is not very effective at removing salts, sodium and chloride associated with deicing activities percolate down through the vadose zone to the groundwater with little attenuation. Studies have shown that sodium, calcium, bicarbonate, and chloride concentrations actually increase with depth (Pitt and others, 1994).

Groundwater Quality in Karst Aquifers

Karst areas may be characterized by unusual hydrology and landforms resulting from a high degree of secondary permeability in soluble rock. Solutional weathering and erosion are the dominant geomorphic agents of karst terranes, which form in areas of limestone, dolomite, marble, gypsum, and other soluble rocks. Landforms characteristic of well-developed karst terranes include sinkholes, sinking streams, solutionally-enlarged bedrock fractures, caves, underground streams, and springs which may respond rapidly to precipitation and runoff events (White, 1988).

In karst areas, significant groundwater recharge occurs through features such as sinkholes and sinking streams. Sinkholes are enclosed depressions (basins) in the land surface generally formed by the dissolution of underlying bedrock. They function as funnels, directing surface stormwater runoff from the ground surface into karst aquifers. Soils in sinkhole bottoms may be thin or nonexistent. Sinking streams range in size from small ditches with ephemeral flows to relatively large perennial rivers. They may sink through a segment of the stream bed or through a discrete opening (swallet), such as a fracture or cave entrance. Flow of surface water into sinkholes and swallets provides rapid, direct recharge of underlying karst aquifers with little or no attenuation of any transported contaminants.

A significant component of karst groundwater migrates through fractures and conduits (including cave passages) which may contain sensitive subterranean ecosystems. Diffuse groundwater movement through a granular aquifer is commonly measured in m or ft per year. However, the turbulent flow of karst groundwater through conduits and fractures may be measured in km or mi per day, especially during high-flow conditions. Discharge of karst groundwater occurs through springs and resurgent cave streams with flow rates comparable to those of surface streams (Mull and others, 1988). In addition to contributing water to surface streams, springs and cave streams often serve as water supplies for individual households or municipalities.

Despite the ability of karst groundwater to move rapidly through conduits and fractures, contaminants introduced into karst aquifers may persist for long periods. This has been demonstrated by groundwater tracing investigations (McCann and Krothe, 1991; Quinlan and others, 1991; Quinlan and Ray, 1990) in which dyes have been retained and slowly released from the epikarstic zone, which is the area of highly weathered (dissolved) bedrock at the ground surface or at the soil-bedrock interface. Moreover, conduits are typically interconnected with fractures, bedding-plane partings, and less integrated bedrock pores. Conduit water may permeate these adjacent bedrock features, which act as storage reservoirs during periods of high flow (Recker and others, 1988).

It should also be note that groundwater flow in some fractured-rock aquifers may share characteristics with conduit-flow aquifers. For example, areas of soluble rock with solutionally enlarged joints and bedding-plane partings may exhibit direct recharge and rapid groundwater flow without an abundance of "classic" karst features such as sinkholes and caves. The Outer Bluegrass physiographic region of Kentucky is an example. In addition, even relatively insoluble rocks (e.g., basalt and granite) may exhibit groundwater flow characteristics similar to those in karst aquifers if fracture patterns are well developed.

Contaminant transport may also be affected by physical properties of the contaminant. For example, Ewers and others (1991) noted that light nonaqueous-phase liquids (LNAPLs) may move several km (mi) per hour through conduits which are not totally flooded, while migration may be limited or stopped where a conduit becomes completely submerged. Such occurrences have been noted by Crawford (1988) in Bowling Green, Kentucky. Karst aquifers are also capable of transporting contaminants from one surface-water basin to another, as demonstrated by Aley (1988) and Duley (1986).

Despite the abundance of literature dealing with karst groundwater quality, it is difficult to generalize about natural (background) aquifer characteristics or to compare them with the characteristics of highway-impacted karst aquifers. Most studies have been conducted in aquifers which have been impacted by one or more land-use activities. Hoos (1990) reports difficulties assessing stormwater runoff impact on karst groundwater quality because of the influence of additional contaminant sources. Investigations of relatively uncontaminated aquifers (e.g., Werner, 1991), typically address natural constituents of carbonate geochemistry—such as calcium, magnesium, carbonate, and sulfate—rather than constituents associated with highway stormwater runoff. In addition, there is a wide range of permeability, groundwater velocity, and groundwater residence time among karst aquifers and even within different parts of the same aquifer. Spatial and temporal water quality variations can be extreme in karst systems, as shown by the hydrochemical results of Werner (1991). Because of these factors, long-term, site-specific monitoring before, during, and after storm and meltwater events is necessary to fully determine the range and variability of background water quality characteristics in a karst aquifer (Quinlan, 1990).

Impacts of Highways on Karst Aquifers

Although the impacts of highway stormwater runoff have been investigated by numerous researchers, relatively little attention has been directed toward assessing its impact on groundwater in karst areas. As noted by Barrett and others (1993), most studies of highway runoff impacts to groundwater have been conducted in areas with relatively thick soils, which attenuate runoff pollution. However, Betson and Milligan (1985) and Milligan and Betson (1985) note that urban stormwater runoff may have a more significant impact on groundwater quality in karst terranes, where soils may be thin or nonexistent. Moreover, there is little information in the literature to facilitate direct comparison of highway-impacted and ambient groundwater quality in karst aquifers. The limited information regarding highways in karst areas is reviewed below.

Keith and others (1995) report the following ranges of concentrations (in μg/L, except as otherwise stated) for highway runoff contaminants in the karstic Mitchell Plain of Indiana: total chromium (11 to 94), total copper (13 to 80), total lead (4.1 to 89), total nickel (8.9 to 69), total zinc (100 to 310), dissolved copper (below detection limit to 9.9), dissolved zinc (4.7 to 25), and suspended solids (69 to 2300 mg/L).

In a study of urban stormwater runoff in Knoxville, Tennessee, Betson (1977) reports that water quality was not significantly degraded, compared to that in a rural area. However, he notes that metals associated with eroded soil are important contaminants. His study suggests that many of the detected constituents are derived from the atmosphere and that urbanization does not appear to contribute significantly to groundwater quality degradation, even in karst areas. His interpretation is that the karst watersheds in his study functioned as contaminant filters or sinks, although he offers little explanation.

Saleem (1977) reports that chloride concentrations associated with highway deicing vary with soil thickness and season in a karstic dolomite aquifer in Illinois. Bradbury and Muldoon (1992) demonstrate a high degree of correlation between highway locations and zones of elevated chloride levels in the karstic dolomite of Door County, Wisconsin. Werner (1977, 1983) also reports increased chloride concentrations in karst groundwater resulting from highway deicing salts.

Garton (1977) discusses groundwater impacts associated with construction of an interstate highway through a West Virginia karst watershed. The aquifer supplies springs used by the Bowden National Fish Hatchery. During storm events, clay and silt from the construction site were transported through the cavernous aquifer, greatly increasing turbidity at the springs. More than 150,000 trout were killed during one storm. Additional fish kills resulted from spills of diesel fuel at the construction site, and construction activities have reduced spring discharge by diverting water away from the karst aquifer. Similar effects have been noted at another West Virginia fish hatchery by Werner (1983), who notes that constructing highways through karst areas increases the risk of hazardous materials impacts to groundwater. Hubbard and Balfour (1993) address potential groundwater impacts from the development of a highway through the recharge area of a cave stream in Virginia. Based on data from Garcia (1992), highway-associated metals (cadmium, lead, nickel, and zinc) do not appear to be a significant component of stormwater runoff in a sinkhole plain at Elizabethtown, Kentucky.

Smith (1991) discusses stormwater runoff, sedimentation, and associated pollutants in the karstic Edwards Aquifer of Texas. He notes that road-related runoff has increased levels of lead, total petroleum hydrocarbons (TPH), metals, and oil and grease. Studies of highway runoff in the karstic Edwards Plateau region of central Texas are being conducted by the University of Texas at Austin and the Texas Department of Transportation (TxDOT, 1992). A consent decree (U.S. District Court, 1990) impacts actions by the FHWA and TxDOT, establishing procedures for future operations within this environmentally sensitive area. The study has resulted in the ongoing development of a bibliography of highway water quality impacts (Barrett and others, 1993), as well as a TxDOT investigation of runoff water quality impacts from highway construction in the Austin area (Malina and Barrett, 1994; TxDOT, 1992). A local ordinance (City of Austin, 1991) also requires transportation-related risk assessments and places limits on impervious cover and additional annual loading of specific pollutants within the sensitive Barton Springs karst watershed.

Numerous reports address environmental and engineering impacts of highways in Tennessee karst areas, including Moore (1981, 1984, 1987, 1988) and Moore and Amari (1987). Documented problems include induced subsidence and collapse, bridging of caves, and flooding and sedimentation of sinkholes. Moore (1984) and Moore and Amari (1987) discuss sinkhole protection in the context of maintaining drainage and flood prevention. Although water quality data are not reported, it is likely that the use of gabions as recommended by the authors has a positive effect on runoff water quality by reducing the amount of sediment and associated contaminants reaching a sinkhole. Additional publications address engineering considerations associated with highway-induced sinkhole collapse in karst areas of the eastern U.S. (Newton, 1984a and 1987), Alabama (Newton, 1976, 1981, 1984b), and Pennsylvania (White and others, 1984).

Problems of karst groundwater contamination are compounded by the use of drainage wells to increase runoff and minimize flooding in areas with inadequate drainage (Hoos, 1990). Removal of soil from sinkholes to facilitate drainage increases contaminant levels in groundwater by reducing the natural attenuation of suspended contaminants. Crawford and Groves (1984) discuss stormwater drainage wells in karst areas of Tennessee and Kentucky. In addition to a brief treatment of well design, they address problems of groundwater contamination and siltation. Stormwater runoff and drainage wells in the urbanized sinkhole plain of Bowling Green, Kentucky, are also considered in other publications by Crawford (1982a, 1982b, 1988).

In a study of urban stormwater runoff in Clarksville, Tennessee, Hoos (1988, 1990) notes that concentrations and loads of some contaminants are significantly less in runoff at a drainage well than in the discharge of a downgradient karst spring, suggesting that the drainage well is only one of several pollution sources. However, some roadway-associated contaminants (arsenic, copper, lead, organic carbon, and oil and grease) are contributed to local groundwater primarily by the drainage well. Trace metals are flushed into the aquifer early during each runoff event. Between February and October 1988, calculated metals loads ranged from 0.030 pounds (cadmium) to 12 pounds (strontium), and nitrogen loads ranged from 0.97 pounds (nitrate as nitrogen) to 34 pounds (organic nitrogen).

Highway Stormwater Runoff Treatment Methods

The primary objective of highway stormwater runoff pollution management is the reduction of total pollutant loads entering groundwater or surface water. Although all highway runoff contains pollutants, their concentrations do not always constitute a problem for the receiving waters. Therefore, runoff treatment must be applied to that portion of the runoff with the highest contaminant concentration. This may be achieved by treating only the "first flush" of runoff, rather than the entire volume.

Numerous techniques exist for treating highway stormwater runoff. Effective methods, which may be implemented individually or in combination, include curb elimination, establishment of vegetation and swales, the use of French drains, Dutch drains, temporary checkdams, sedimentation ponds, detention basins, and filtration. Keith and others (1995) discuss the contaminant removal efficiency of rock and peat filters in sinkholes receiving highway runoff. Preliminary findings indicate that one peat filter has been removing approximately 80 percent of suspended particulate materials and about 50 percent of the dissolved copper and zinc. Rock filters have been removing approximately 33 to 76 percent of the suspended solids and 35 to 55 percent of the total recoverable metals. However, rock filters do not appear to remove dissolved metals.

Significant reductions in pollutant loads from highway runoff can result when some general guidelines are followed (Maestri and Lord, 1987). Treatment systems using swales and infiltration should be designed to maximize contact with the soil. Because dissolved hydrocarbons appear to be a significant component of highway stormwater runoff (Asplund and others, 1980; Zawlocki and others, 1980), sedimentation and/or filtration may be inadequate for removing all the pollutants from runoff. Adsorption or other removal methods may be required.

EVALUATION AND SELECTION OF FIELD TESTING SITES

Two sites will be used for field testing of the highway runoff treatment methodology developed during this study. The ideal site would be a sinkhole (within a right-of-way) which receives large quantities of runoff derived exclusively from a high-ADT highway. It would have a direct connection to a karst aquifer with little opportunity for natural attenuation by sedimentation or percolation through soil. Finally, there would be existing documentation regarding possible downgradient groundwater quality monitoring locations, such as cave streams and springs.

During the initial phase of this investigation, PELA conducted site visits to evaluate locations where highway runoff potentially impacts groundwater in karst areas. More than 127 sites were visited in the fifteen states participating in this study. Additional information was obtained from Department of Transportation (DOT) personnel and others with recognized knowledge of the karst areas in each state or region. These data were used to assess the suitability of each site for use during field testing of the runoff treatment method.

Three sites have been determined to be the most suitable for field testing of runoff treatment methodology. They are located at I-40/I-640 in Knoxville, Tennessee; I-65 in Cave City, Kentucky; and I-70 in Frederick, Maryland. The Knoxville site appears to provide the best combination of desirable factors—high ADT, right-of-way location with sufficient work area, large proportion of highway-derived runoff, direct recharge of a karst aquifer with minimal natural filtration, and documented groundwater flow routes and monitoring locations. The selection of an additional site (in Kentucky or Maryland) will depend on factors still being assessed, such as highway runoff water quality, cooperation of a property owner at the Maryland site, and the ability of each DOT to provide assistance.

SENSITIVE KARST AREAS OF THE UNITED STATES

One objective of this project is the identification of sensitive karst areas in the U.S. A map has been prepared to delineate karst regions throughout the forty-eight contiguous states, including areas with high potential for groundwater contamination and sinkhole collapse. (This map could not be produced readily at a scale suitable for inclusion in this paper.) Additional maps have been prepared to depict sensitive karst areas within the fifteen states participating in this study. The maps are based on data presented by Davies and others (1984) and Davies and LeGrand (1972) with modifications based on information from a variety of publications and contacts with local experts.

INVESTIGATION PLAN

The primary objective of this project is to design and test a method for improving the quality of highway runoff draining into karst aquifers through sinkholes. The treatment methodology will involve a combination of sedimentation, filtration, and adsorption designed to treat the first flush of stormwater runoff. The first phase of testing will consist of laboratory experiments to determine the most effective combination of filter media, thickness, and compaction. These trials will be conducted by the UT Civil and Environmental Engineering Department, through the auspices of the Institute for Geotechnology.

Based on the literature review, a peat-sand filter (PSF) should provide good water-quality enhancement for highway stormwater runoff. A PSF should exhibit good contaminant removal, simple design, and minimal construction, maintenance and operating costs. Good phosphorus, heavy metal, BOD, and pathogen removal capabilities, coupled with a simple design and low maintenance, make the PSF methodology attractive for treating highway runoff. The PSF system is a hybrid filtration system combining the attributes of a peat filter/adsorption bed with a nutrient-removing grass cover crop and subsurface sand layer(s). This combination has been shown to achieve a high overall pollutant removal efficiency within a single, relatively compact unit.

The planned laboratory study includes testing of hemic (reed-sedge) peat. Hemic peats have intermediate values of hydraulic conductivity, bulk density and water-holding capacity (MacFarlane and Radforth, 1986). Soil preparation may include preliminary conditioning, such as shredding to approximate uniform density (Stanlick, 1976). A Proctor test will be run to determine optimum water content for compaction and compactive effort necessary for desired hydraulic conductivity. Proctor tests are not typically performed on peat and, therefore, information on the range of optimum water content is limited. The compactive effort necessary to achieve the desired hydraulic conductivity to provide efficient treatment and drainage is approximately 0.25 in/hr (MWCOG, 1987). This compactive effort may require adjustment to insure adequate support for maintenance equipment. The filter also requires a layer of fine- to medium-grain sand and a layer of pea gravel to prevent clogging.

Pilot treatment structures will be installed and tested in two of the sinkholes selected for evaluation. The hydrologic function of the structures will be monitored in the field. Water samples collected before and after treatment will be chemically analyzed to evaluate water quality improvement. The hydrologic performance efficiency will also be monitored to assess the long-term viability of the structures and projected maintenance requirements. Prior to installation of the test structures, baseline data will be collected on the discharge and quality of highway runoff draining into each sinkhole. This information will facilitate the design of the structures and will provide the data necessary to assess water quality improvement associated with their use.

The pilot structure will be designed to treat primarily the first flush of runoff, as determined by the baseline investigation. To prevent flooding at the site, the structure will be designed to permit subsequent runoff to bypass the treatment process and flow directly into the sinkhole. It is anticipated that runoff water will be pretreated in a preliminary detention basin to promote settling of coarse sediment and removal of floating debris. The treatment media will probably be placed in the flat bottom of a large, shallow retention basin. A perforated drainage pipe beneath the media will collect treated drainage and conduct it into the sinkhole. The surface of the treatment media area will be designed to minimize clogging and to facilitate maintenance.

The drainage conduit from the treatment structure into the sinkhole will be designed to permit sampling of the treated runoff as it recharges the karst aquifer. Analytical results from these samples will be compared with those collected upstream of the treatment structure to test its effectiveness. Samples will be collected to monitor the treatment of the maximum contaminant concentration during the first flush. Because of the time lag resulting from detention and treatment of the runoff, the total load of each contaminant before and after treatment will be compared, if feasible. If appropriate, samples will also be collected at the spring to assess water quality improvement at the aquifer scale.

CONCLUSIONS

Stormwater runoff transports solids, heavy metals, nutrients, bacteria, road salt, herbicides, and hydrocarbons from highway surfaces as a result of normal operation and maintenance, as well as from atmospheric deposition. A large portion of highway runoff pollutants are associated with particulate material. Highway drainage generally transports most pollution during the "first flush" of runoff from a storm event. Factors which affect contaminant levels in highway stormwater runoff include precipitation and traffic characteristics, as well as surrounding land use patterns. Highway-related impacts to water quality appear to be most significant from highways with traffic in excess of 30,000 vehicles per day. Other factors include proportion of impervious area, pavement type, drainage mechanisms, right-of-way vegetation, precipitation characteristics, and construction activities, as well as maintenance and deicing policies and regulations regarding automobile emissions, maintenance, and studded snow tires.

The effects of highway runoff on groundwater quality depend on local hydrogeologic conditions, including sorption processes of aquifer materials and groundwater velocity. Because of the near-surface attenuation of pollution, highway runoff may pose a greater threat to soil water than groundwater in many areas. Particulate metals may be filtered out at the soil surface, and dissolved metals may be removed by adsorption onto soil particles during infiltration. However, highway runoff may have a more significant impact on groundwater quality in karst terranes, where soils may be thin or nonexistent and where groundwater recharge may occur directly through fractures, sinkholes, and sinking streams. While hazardous materials releases present the greatest risk of acute groundwater impacts in karst areas, chronic or periodic impacts have been documented from highway-related particulate materials, heavy metals, chlorides, and oil and grease. Whenever possible, highway alignment and other engineering controls should be designed to minimize the runoff of highway stormwater drainage into karst aquifer recharge features (sinkholes). Runoff treatment methods should be used only in areas where there is no feasible alternative to subsurface disposal of stormwater.

Because most contamination in highway stormwater runoff is associated with particulate material, sedimentation and/or filtration should be included in any treatment system design. However, because dissolved hydrocarbons appear to be a significant component of highway runoff, adsorption or other measures should also be considered.

Following field visits to fifteen states, suitable sites for field testing of the runoff treatment methodology have been located in Knoxville, Tennessee; Cave City, Kentucky; and Frederick, Maryland. At each of these sites, a karst aquifer receives recharge directly from high-ADT highways and related surfaces. Hydrologic connections between each site and a downgradient spring (or springs) has been established by previous dye-tracing investigations.

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LANDSLIDE REPAIR UTILIZING A STEEPENED GEOGRID REINFORCED SLOPE, INTERSTATE 90 M.P. 28.9, SHERIDAN COUNTY, WYOMING

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ABSTRACT

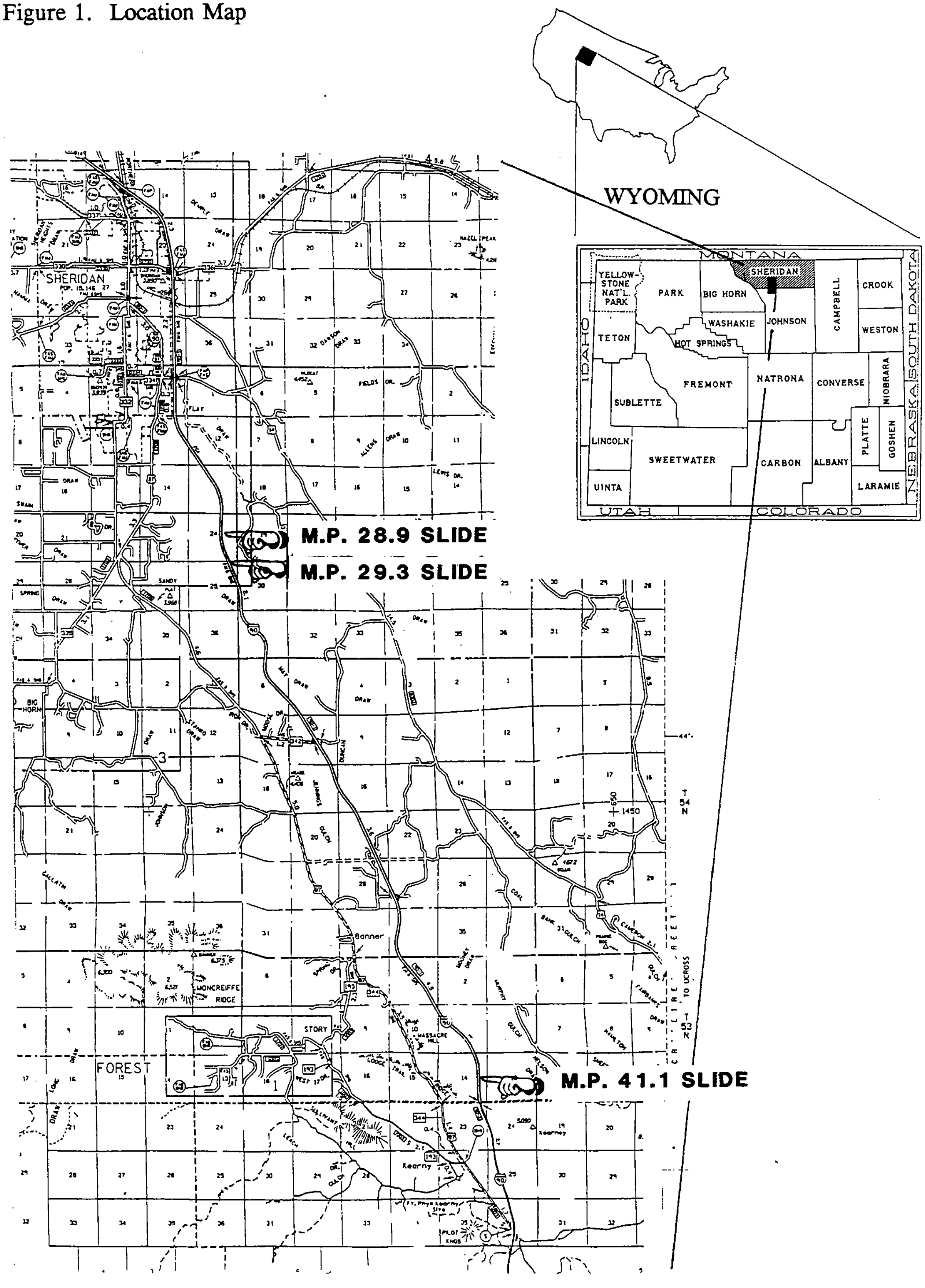
The largest geogrid reinforced slope constructed in Wyoming to-date was built in the summer of 1994 as part of a project to repair a landslide which was occurring in an 80′ high embankment in the southbound lane (SBL) of I-90. This embankment was built in the 1960's and was constructed from the native, highly plastic, clay soils. Movement of the slide, which affected approximately 300′ of the roadway, began in 1987 and had become a continual maintenance problem. From a detailed geotechnical investigation in late 1992, it was determined that there was no easy remedial solution to prevent further movement of the slide. It was decided that to prevent catastrophic failure, the best solution would be to shift the SBL approximately 100′ eastward toward the northbound lane (NBL) to reduce the driving force on the slide.

Even with the proposed line shift, the toe of the 2:1 fill slope still would have been in active slide material. The safety factors from the XSTABL program for this slope configuration were unacceptable. In order to get the toe of the new fill out of the active slide area, a slope of 1.5:1 was required. To get this 1.5:1 slope, it was decided to reinforce the native soil with geogrid. In order to do the line shift, the existing 80′ high embankment had to be removed and up to 35′ of soft foundation material had to be excavated to key the new embankment into bedrock. Approximately 27,000 yds^2 of uniaxial rigid geogrid were used to reinforce the new embankment which was constructed from the 211,000 yds^3 of material that was removed from the existing fill.

As part of the line shift through the slide area, the adjacent existing fills were being widened to bring the roadway template up to current design standards. During construction, when widening of a similar fill approximately 2000′ south of the active slide area was completed, a fill section 200′ long and 100′ high began to fail. A quick XSTABL analysis was run based on the location of the cracks in the roadway and the bulge at the toe of the slope. This rough XSTABL analysis indicated that a berm at the base of the slope could stabilize the slide. Within hours, the contractor began to build the toe berm and movement of the slide was stopped.

The Geology Program of WYDOT mobilized a drill rig to do a complete geotechnical investigation of the slide. The investigation determined that the slide plane which the XSTABL program had predicted was quite accurate. The size of the toe berm then was increased to obtain the desired safety factor. The cost of this stabilization was approximately \$40,000. Based on the previous contract costs for a similar slide repair, it is estimated that if this slope had completely failed, it would have cost \$500,000-\$600,000 to repair.

Figure 1. Location Map



INTRODUCTION

Interstate 90 is the primary east-west Interstate Highway across the northern portion of Wyoming. I-90 crosses or skirts the Powder River Basin for the majority of its length through the state. Figure 1 is a general location map of the slide area. The primary formation that outcrops along this route is the Tertiary Wasatch Formation consisting of interbedded shales, coal, and sandstone. There are coal beds in the formation that are over 200' thick. In the Sheridan area, the Wasatch consists of clay shale and thin coal seams. Most of the thin coal seams carry water and create "perched" water tables where lateral discontinuities such as faults are encountered. The water from the coal seams is a major factor that contribute to the countless slides and surface slumps that can be observed throughout the area.

MAIN SLIDE M.P. 28.9

The 80' high embankment at milepost 28.9 in the SBL of I-90 was constructed in 1964. In 1987, cracks began to form in the roadway in a section approximately 300' long . At that time, one

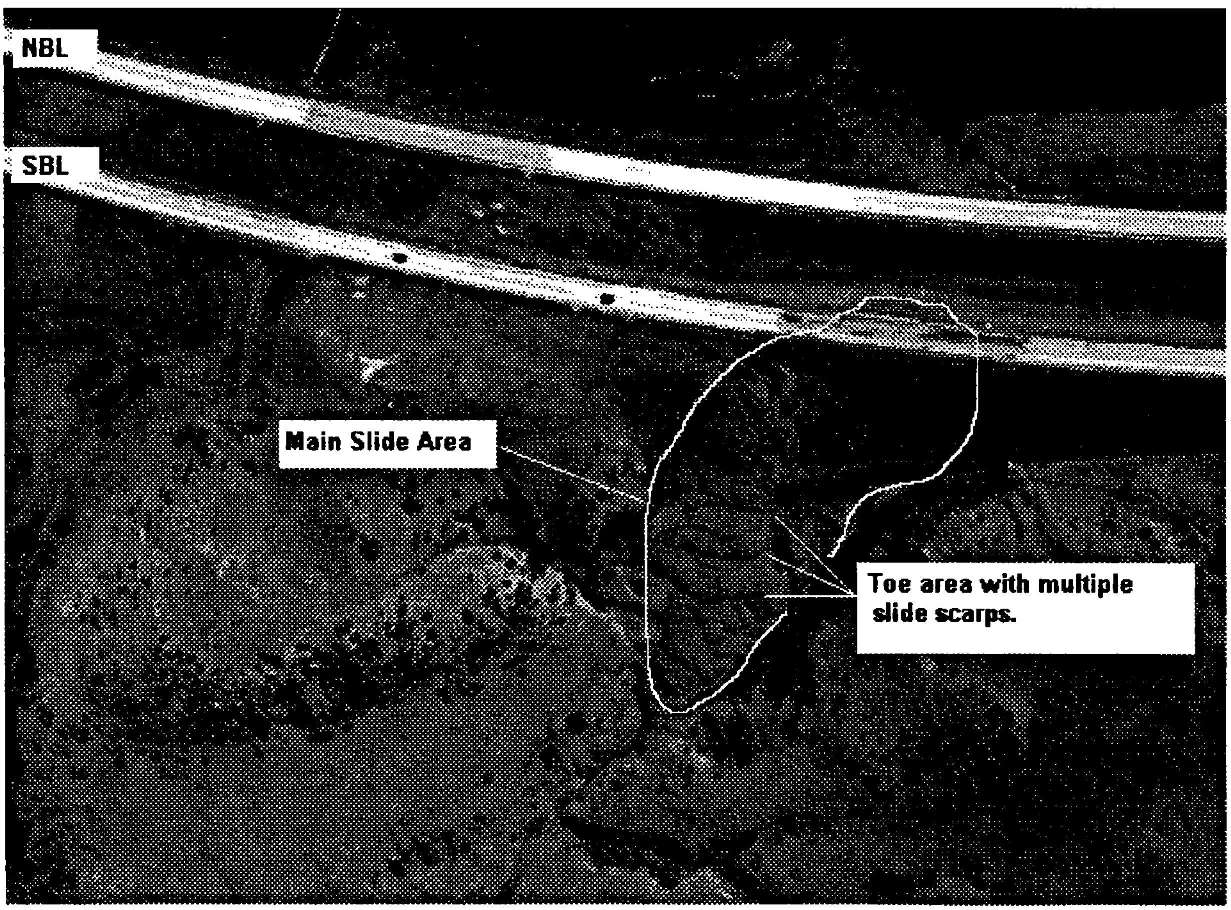


Figure 2. Air photo of M.P. 28.9 slide before reconstruction.

test hole was drilled and a PVC pipe was installed to monitor the groundwater level and possibly detect the depth of the slide plane. Settlement continued through the area and the PVC pipe was covered with an asphalt patch before the depth of the slide plane was determined.

In 1992, the settlement of the road accelerated and circular cracks could be traced along the fill slope to the base of the embankment. The cracks extended completely across the SBL and partially into the fill slope on the median side. A detailed geotechnical investigation of the slide began in late summer 1992.

At this site, the NBL and the SBL are separated by approximately 200′, and the grade of the NBL was approximately 10' above the grade of the SBL. Figure 2 is an air photo of the site. As shown in the photo, the material in the drainage at the toe of the embankment consists of multiple slump blocks. The slump blocks extend approximately 500′ down the drainage from centerline. The deep erosion rills that can be seen on the fill slope are due to surface water flowing down the grade of the roadway and diverting down the fill slope in the area where the slide was affecting the roadway.

Figure 3 is a plan view of the test hole locations with the outline of the slide limits shown. The greatest amount of visible surface movement was evident along the north side of the drainage near the outlet end of the 42" CMP. In this area there were numerous new surface slumps and the right-of-way fence had been replaced because the movement of the slide had displaced it.

The subsurface conditions encountered during the investigation are shown in Figure 4. The typical material under the roadway of the SBL consisted of 69″ of surfacing (52″ asphalt and 17″ base) overlying 51′ of stiff, moist, silty clay fill with shale and sandstone fragments. The natural ground under the fill contained up to 30′ of alluvial slope wash material which consisted of clayey silt with small fragments of scoria, coal and sandstone in the lower 5′ of the layer. In most of the test holes, groundwater was encountered in the bottom 2′-3′ of the slope wash.

Continuous soil cores were taken in all of the test holes. From the jumbled, distorted portions of the cores it was possible to identify the slide plane. An XSTABL analysis was performed utilizing drill hole data, and the analyzed slide plane is shown in Figure 4. The majority of the analyzed slide plane length is through the slope wash material. See Table 1 for soil parameters.

TABLE 1

SOIL TYPE	COHESION (PSF)	PHI (DEG)
Slope Wash (Unsaturated)	838	9
Slope Wash (Saturated)	500	0
Clayey Silt	500	5

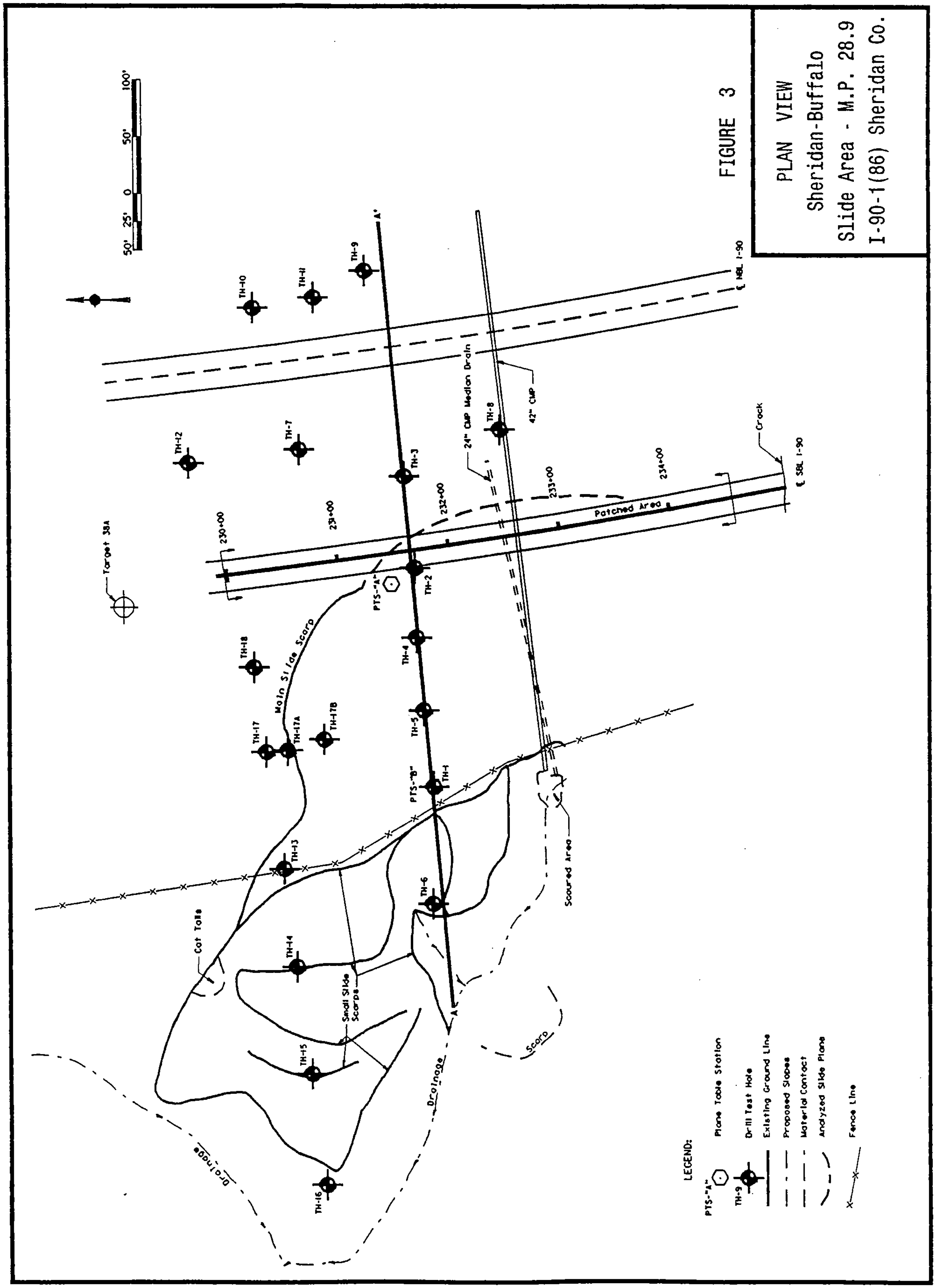


FIGURE 3

PLAN VIEW
Sheridan-Buffalo
Slide Area - M.P. 28.9
I-90-1(86) Sheridan Co.

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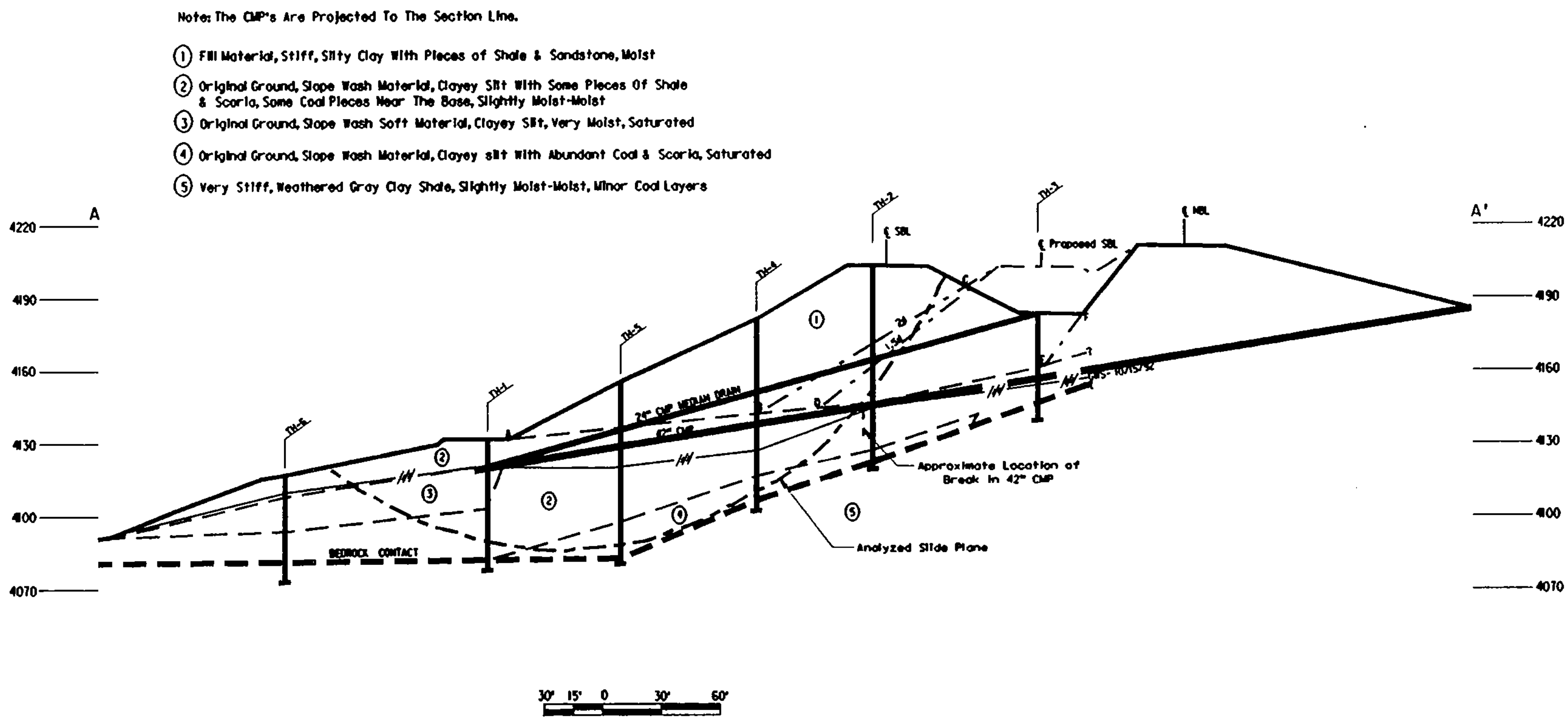

FIGURE 4

X-SECTION A-A'
Sheridan-Buffalo
Slide Area - M.P. 28.9
I-90-1(86) Sheridan Co.

After the completion of the investigation and the XSTABL analysis, it was evident that the slide intersected the 42″ CMP which provided the drainage through the embankment (see Figure 4). A field inspection of the pipe revealed that the pipe had been separated and was partially collapsed by the slide movement approximately 200′ in from the outlet. This rupture location in the pipe was very close to the slide plane that the XSTABL program had predicted. The discovery of the ruptured pipe made it imperative that some type of corrective action be taken to repair the slide. All of the surface water was being channeled into the unstable material, further aggravating a deteriorating situation.

A memo emphasizing the potential for catastrophic failure and loss of the entire SBL through this area was forwarded to District personnel in February of 1993. As a result of this memo, temporary cross-overs were built and an emergency traffic plan was developed to provide for two-way traffic in the event the SBL failed.

Many options were considered during the design of the slide repair. The first option considered the use of a toe berm to add resisting force to the slide. Since the toe of the embankment was at the head of a series of small slumps in the drainage, and the foundation material at the toe was very soft and saturated, building a toe berm would have resulted in shear failure of the toe. The use of lightweight fill to reduce the driving forces was also considered, however the size of the fill made it impractical and cost prohibitive with over 50% of the fill replaced with lightweight material. Lowering the grade of the SBL was also considered, but lowering the grade enough to increase the safety factor significantly would have made the lane too steep for Interstate highway design standards.

Different methods of subsurface drainage were also considered, including: horizontal drains installed in the toe of the slope; a deep cutoff drainage curtain in the median; and a series of deep wells to lower the groundwater. Due to the inconsistent nature of the subsurface water, it was decided that none of these methods by themselves would raise the overall safety factor of the slide significantly enough to justify the cost.

Since most of the embankment was over the active portion of the slide, the most effective way to reduce the driving force along the slide plane was to remove the embankment and realign the SBL approximately 100′ toward the NBL. This realignment kept the same grade but required the reconstruction of approximately one mile of the SBL. The removal of the existing embankment also provided access to repair the portion of the 42″ CMP that had been damaged. Even with the 100′ alignment shift, if the new embankment was constructed at a 2:1 slope, the toe and approximately one-third of the new fill would have been placed over the slide plane. For this reason, it was decided to key the new fill slope into bedrock and steepen it to a 1.5:1 slope through the use of geogrid reinforcement.

During the excavation of existing fill, the slide plane was exposed at many different places. The most dramatic evidence of the slide plane was seen in a trench that was dug parallel to the 42″ CMP near the point that the CMP had been ruptured by the slide. Figure 5 shows the slide plane and the wide separation of the two soil masses at this location.

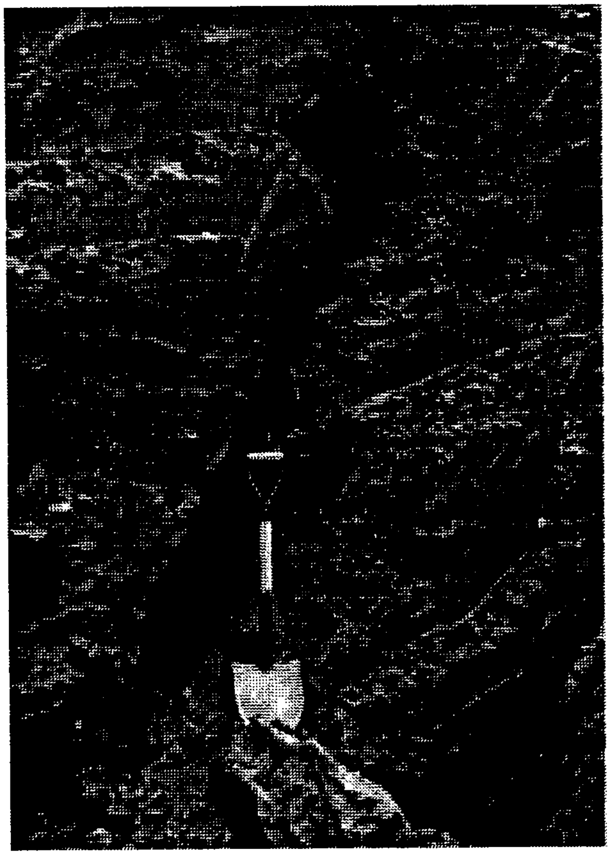

Figure 5. Photo of the slide plane after it was exposed during the excavation of the 42″ CMP. The slide plane was a large open void which could easily collect water and decrease the strength of the clayey soils.

Figure 6 shows the typical geogrid installation in cross-section and elevation view. Nine layers of rigid, uniaxial geogrid were used to reinforce the slope. The embedment length of the layers averaged approximately 95′. The bottom grids were spaced at 5′ intervals and the grids in the upper layers were installed at 8′ intervals. Approximately 27,000 yds^2 of geogrid were used. The embankment was built from material that had been excavated from the original fill. This material consisted of clay and pieces of shale which classified primarily as A-6 and A-7-6 soils. Approximately 211,000 yds^3 of material were required to build the reinforced fill.

At the base of the excavation where the new fill was keyed into bedrock, a drain system was installed to collect the water that was seeping out of the numerous coal seams and highly fractured scoria. Scoria backfill was used in the base of the excavation to add friction between the shale bedrock and the new embankment and also as a backup system to provide drainage if the perforated pipe underdrain system failed.

One of the major concerns during the construction of the embankment was the stability of the NBL. The excavation for the SBL was benched on a 1.5:1 slope, which was approximately 100′ vertically from the edge of the NBL to the base of the excavation. An XSTABL analysis of this slope configuration resulted in a safety factor of 1.2. It was decided that this was an acceptable safety factor for the one-month time period that the excavation would be open.

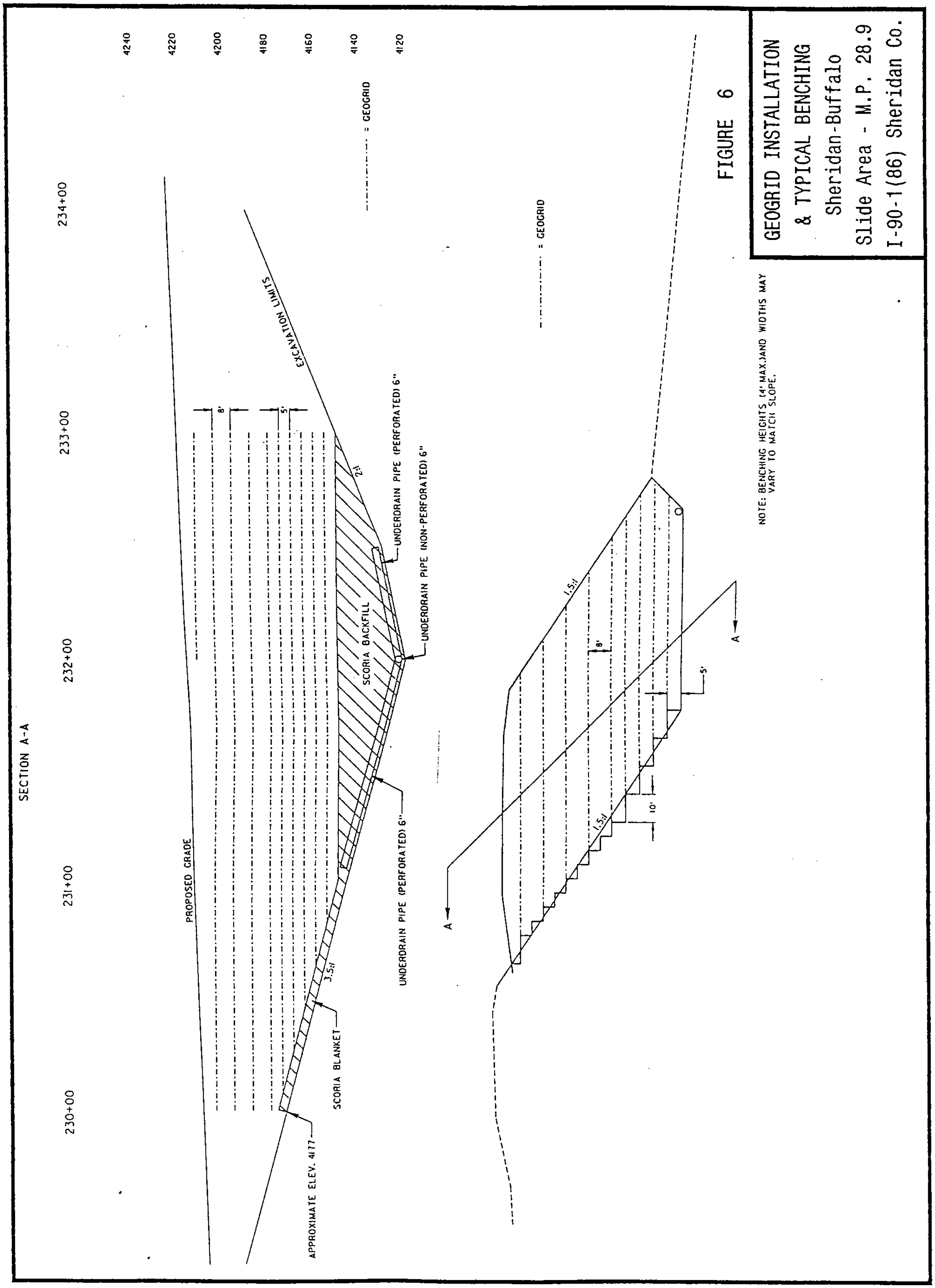

FIGURE 6

GEOGRID INSTALLATION
& TYPICAL BENCHING
Sheridan-Buffalo
Slide Area - M.P. 28.9
I-90-1(86) Sheridan Co.

Figure 7. Excavation of South Bound Lane and benching on North Bound Lane.
Photo looking south on centerline of South Bound Lane.

During construction it was the traffic control contractor's responsibility to monitor the NBL for any signs of instability since they were on the project 24 hours per day. Fortunately, no stability problems developed during the time the excavation was open (see Figure 7.)

CONSTRUCTION SLIDE M.P. 29.3

As part of the line shift through the slide area, the fills through the realigned area were widened to bring the roadway template up to current design standards. Figure 8 is a typical cross-section showing the "sliver" fill widening. The new fill in these areas was tied to the existing fill with benches approximately 10′ wide.

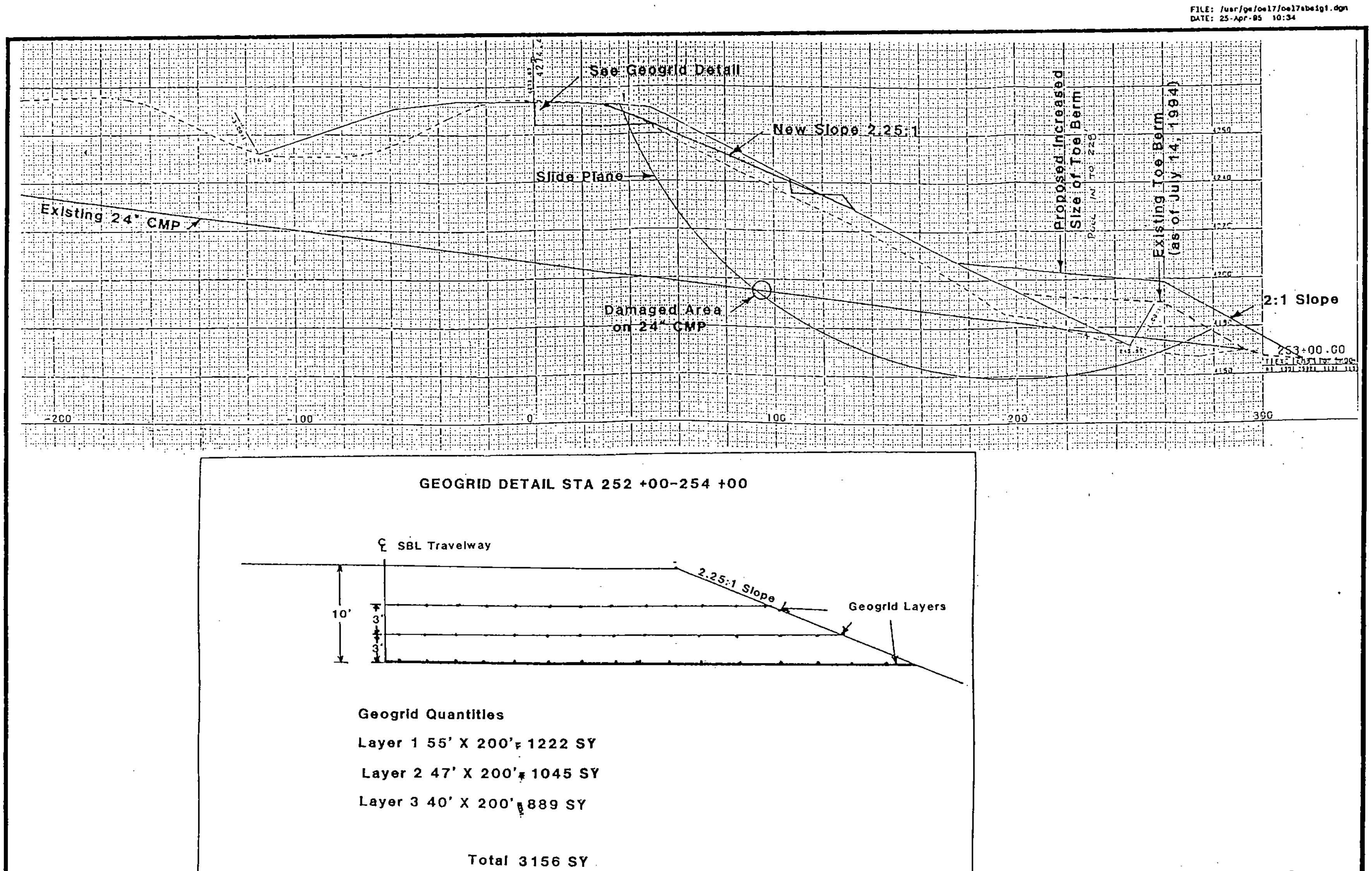

Figure 8.

MP 29.3 Construction Slide
I-90-1(86)
Sheridan-Buffalo
Sheridan Co.

On the morning of June 29, 1994, two days after the "sliver" fill (2000′ south of the main slide) was complete, cracks were discovered in the roadway (see Figure 9). The arcuate cracks affected a 200′ length of the roadway to approximately the center of the driving lane and could be traced to the bottom of the slope on both ends. A bulge approximately 2′ high had formed at the base of the fill slope where the toe of the slide had ruptured.

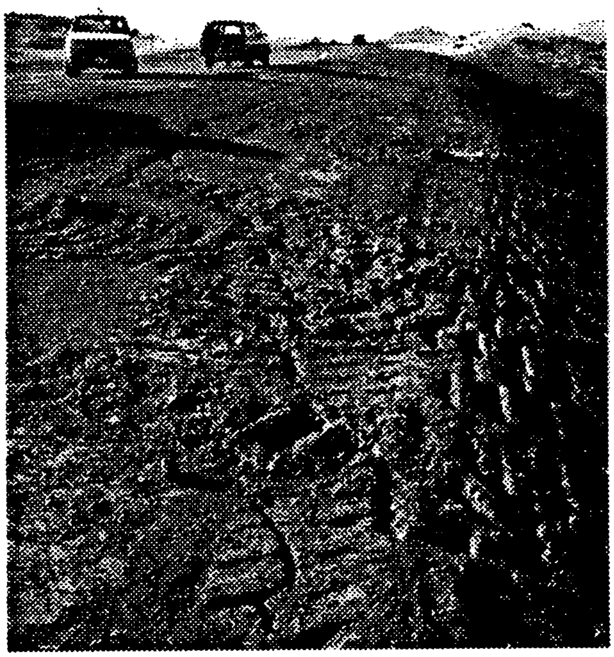

Figure 9. M.P. 29.3. Cracks in the embankment just after completion of the sliver fill.

The location of the cracks and the bulge were quickly measured and the information was phoned to the WYDOT Geology Program in Cheyenne. A random XSTABL analysis using this information was performed immediately, and a circular slide plane which extended below the base of fill and surfaced beyond the toe of the fill, was identified as the most critical surface (see Figure 8). Based on the geometry of this slide plane, it was determined that a toe berm would add enough resisting force to stop the movement of the slide.

The biggest challenge, after the decision of what remedial action should be taken, was to get the toe berm built before the slope failed. By afternoon of June 29, the main crack on the roadway had opened up to approximately 3″ wide. By late afternoon on the 29th, the contractor had begun to move dirt to build the toe berm. The construction of the toe berm required the extension of a 24″ CMP and three underdrain outlets at the toe of the slope. By July 1, the emergency toe berm was completed and the movement of the slide had ceased. It was calculated that this berm increased the safety factor of the slope to 1.2.

Also on June 29, construction of a similar "sliver" fill had begun on the north end of the project. After seeing the failure that the addition of the "sliver" fill had caused at M.P. 29.3, it was decided to halt construction of the rest of the proposed fill extensions on this project.

During the week of July 5, a complete geotechnical investigation by WYDOT'S Geology Program was performed. Continuous soil cores were taken and the slide plane was identified in all of the test holes. The slide plane that was identified in the drill holes was nearly identical to the most critical surface that the XSTABL program identified.

Using soil parameters obtained from samples taken during this investigation, it was determined that the safety factor of the fill slope before the addition of the "sliver" fill was 1.02. The weight the "sliver" fill added was enough to drop the safety factor to below 1.0 and cause the fill to begin to fail. From the results of this analysis, the delicate stability of these large clay fills was realized.

To obtain an acceptable safety factor of 1.5, which is WYDOT's standard for Interstate highways, it was recommended that the height of the emergency toe berm be increased 10′. In addition, it was recommended that the "sliver" fill that had been placed through this area be removed and the slope returned to its original configuration. Approximately 18,000 yds^3 of material was required to build the toe berm (see Figure 8.).

The next problem that was considered was the possibility that the movement of the slide had ruptured the 24″ CMP that provided drainage under the fill. If the pipe was ruptured, the surface water would be channeled directly into the fill and potentially create further instability. Since this pipe was such a small diameter, approximately 600′ long and installed at a 14% grade, it was impossible to see from one end to the other to determine if it was ruptured. A geologist from WYDOT's Geology Program with an interest in caving, agreed to do a visual inspection of the inside of the pipe. From this inspection, it was determined that the 24″ CMP had been separated and partially crushed 205′ from the outlet end. This is the exact location that the slide plane crossed the pipe in the XSTABL analysis. Since the surface area that this pipe drained was small, it was possible to re-contour the drainage area so the water flowed away from the pipe inlet. The ends of the pipe then were sealed to prevent any further infiltration of water into the fill.

The cost of the slide stabilization, including the toe berm, changing the drainage, and sealing off the 24″ CMP, was approximately $40,000. Based on previous contract costs for similar slide repair, it is estimated that, had this slope completely failed, it would have cost $500,000-$600,000 to repair.

CONCLUSIONS

The main slide repair at M.P. 28.9 was very successful. The construction of the 80′ high geogrid reinforced slope was the largest reinforced slope that had been built in Wyoming. It was also the first time in Wyoming that a slide of this size was repaired before it completely failed. The repair of this slide illustrates the changing trend at WYDOT to become proactive to these types of potential slope failures instead of reactive to complete slope failure. This is becoming increasingly important in this time of budget cutbacks. The slide repair was combined with a five mile long overlay project which helped reduce the mobilization and construction engineering costs.

In a strange way, it was fortunate that the construction slide at M.P. 29.3 occurred when it did. Since the dirt contractor was still on the project, the cost of repairing the slide was significantly reduced since there were no mobilization costs. It allowed for the berm to be built before the slope could fail completely, and it also prevented the construction of other "sliver" fills on this project which had a high potential to fail.

The lessons that were learned from this slope failure will be very valuable in the future. Approximately 25 miles of I-90 are planned for upgrading, which will require widening of similar large fills. At the present time, the options being considered to prevent the placement of the "sliver" fills are: filling in the medians and widening both lanes toward the media; or lowering the grade through the fill areas to obtain the required roadway template widths.

The importance of continual maintenance was also illustrated during the construction of this project. In two of the large fills, the outlet ends of two drainage pipes were uncovered. At one site 8′ of material had covered the outlet, and at the other site 17′ of material blocked the outlet. Both pipes were completely full of water, which was causing continual saturation of the toe of these large fills. It has been recommended that District IV Maintenance personnel increase their efforts to maintain proper surface drainage through these critical areas.

It appears that the construction and maintenance of the large clay fills in this region will provide many geotechnical challenges for WYDOT for many years to come.

ACKNOWLEDGEMENTS

The authors would like to thank Connie Fournier and Marilyn Foster for typing and editorial assistance, Jim Dahill for all of the field work and XSTABL expertise, Bret Boundy for his caving ability, and all of the District IV personnel who cooperated to make this project possible.

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DESIGN OF ROCK REINFORCEMENT FOR THE TALLULAH GORGE BRIDGE FOUNDATIONS

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ABSTRACT

Georgia Route 15 (U.S. 23), across the Tallulah River Gorge in Northeast Georgia, was constructed in 1939 as a two lane bridge approximately 505 feet long. The bridge was built immediately downstream of the existing Tallulah Falls Dam, a concrete gravity structure with a spillway forming the crest which was completed in 1913.

In 1989 the Georgia Department of Transportation proposed to widen the existing bridge to four lanes utilizing the existing foundation piers. A study was conducted to evaluate the stability of the foundation rock mass in the vicinity of and below the pier foundations as a result of the proposed widening of the bridge.

Geologic mapping and collection of other field data was complicated by the steep terrain and difficult access to the gorge. Mapping and survey, using rapelling techniques and rock coring utilizing a gas powered, portable drill rig were performed. Specially designed laboratory direct shear testing was performed to evaluate shear properties along planes of weakness in rock samples. A commercially available computer program was utilized to create a data base of rock parameter information and create stereograph projections of rock discontinuities across the site.

Rock slope/wedge stability analyses were performed for different areas in the vicinity of pier foundations to evaluate stability of the foundations under the new loads imposed by bridge widening. Wind and earthquake loading assumptions were included in the analyses.

Based on the results of the analyses and visual examinations of the rock in the vicinity of the foundations, a program of systematic rock reinforcement was designed to achieve required safety factors against sliding and to maintain the surrounding rock blocks in position. Under the existing bridge pier foundations, pier anchor tendons were designed to tie proposed new columns to the underlying foundation.

INTRODUCTION

The Tallulah Gorge Bridge is located at the town of Tallulah Falls in southern Rabun County, Georgia. The bridge is part of an important highway link between Atlanta and the Northeast Georgia Mountains resort and recreation area. The existing two-lane bridge was constructed to carry Georgia Highway 15 across the Tallulah River (Photo 1). It is approximately 505 feet long and supported by two piers, each about 90 feet high (Figure 1). The piers rest on benches excavated into the rock slopes on either side of Tallulah Gorge (Photos 2 and 3).

Located approximately 70 feet upstream of the bridge is Georgia Power's Tallulah Falls Dam. The dam is a curved concrete gravity structure with a gated spillway forming the crest (Photo 4). The spillway extends to the bottom of the gorge between and below the elevation of the bridge piers. Under certain reservoir conditions water is released over the spillway and into contact with rock adjacent to and below the pier foundations.

The Georgia Department of Transportation commissioned a study of the bridge pier foundations and a geotechnical analysis of the underlying rock mass in preparation for widening of the existing bridge from two lanes to four. No historical information regarding conditions at the pier foundations at the time of construction was available for review.

PHYSIOGRAPHY AND GEOLOGY

The bridge site lies within the Blue Ridge Physiographic Province whose topography is characterized by high ridges, steep slopes and narrow valleys. The Tallulah River flows through a natural gorge that extends from the Tallulah Falls dam southeastward to the confluence of the Tallulah and Chattooga Rivers. At its deepest point the gorge is about 700 feet deep. At the bridge site the gorge is about 150 feet deep.

Slopes within the study area are steep ranging, from about 40 degrees to vertical. The upper slopes of the gorge typically consist of discontinuous outcrops of weathered rock covered or partly covered by a thin layer of sandy soil. In the vicinity of the pier foundations continuous outcrops of competent, but jointed and fractured rock are common. The bottom of the gorge is characterized by exposures of relatively solid competent rock.

Geologically, the study area lies within the Tallulah Falls Dome, a large anticlinal structure about 9 to 15 miles in diameter. The core of the dome consists primarily of the Tallulah Falls Formation (Hatcher, 1971). Rocks at the site are typically light brown to gray, medium to coarse grained, foliated quartzite. Thin, discontinuous lenses of muscovite-biotite schist and graphitic-micaceous schist occur parallel to foliation throughout the site. Foliation typically dips about 20 to 25 degrees across the site.

In addition to well defined foliation, other features that were observed to interrupt the continuity of the rock mass included: fractures, joints (more or less continuous fractures along which no movement was apparent), faults (fractures along which movement was apparent or suspected) and mineral veins. Specific information about these discontinuities was collected

PHOTO 1 - TALLULAH GORGE BRIDGE AND TALLULAH FALLS DAM

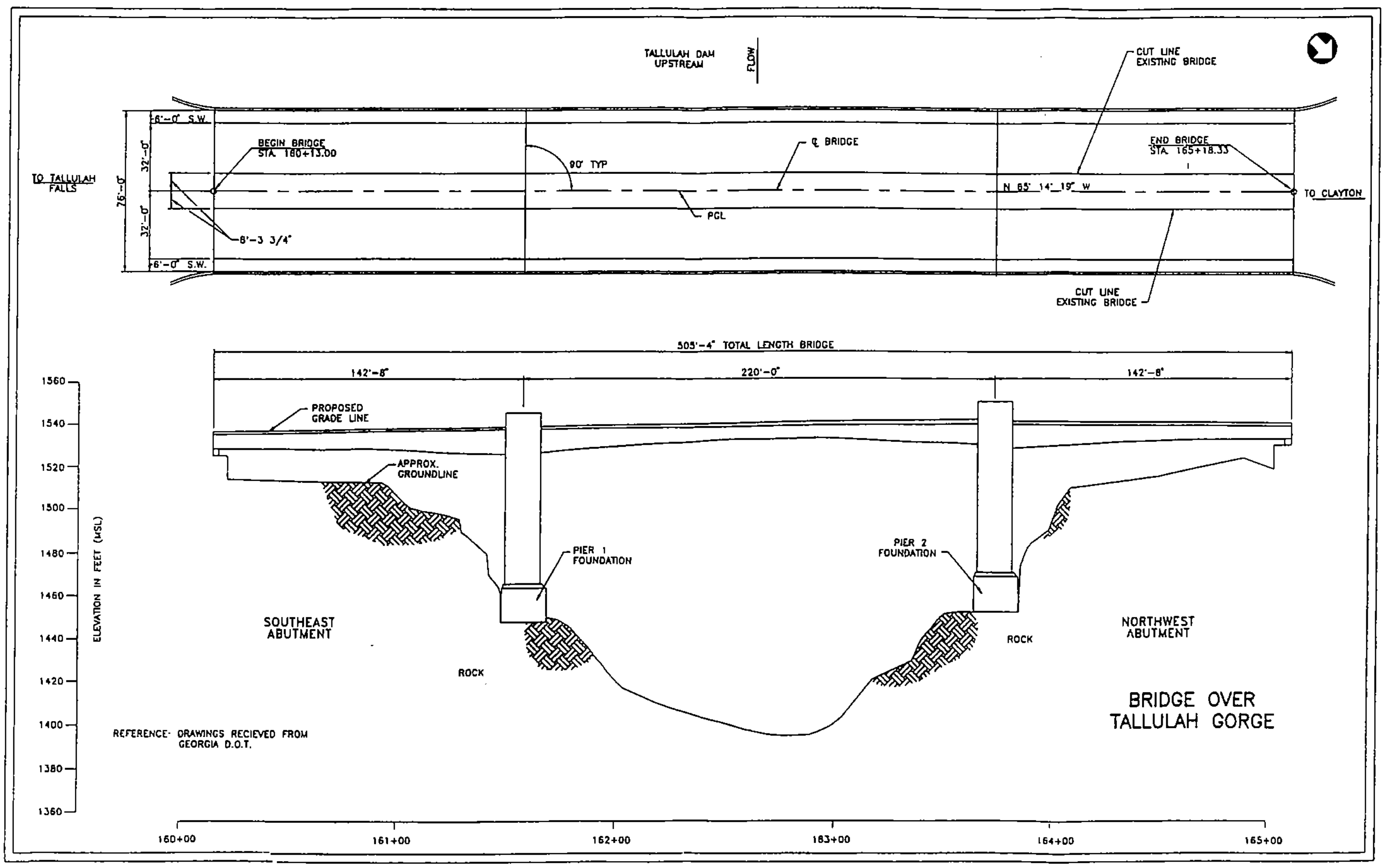


FIGURE 1

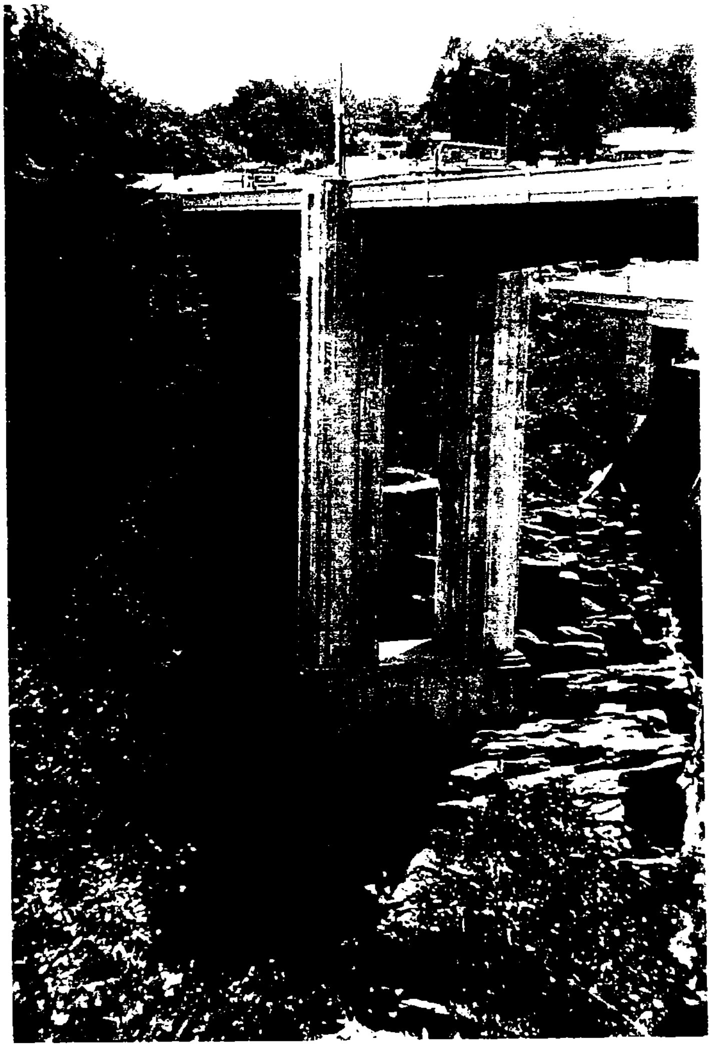


PHOTO 2 - PIER 1

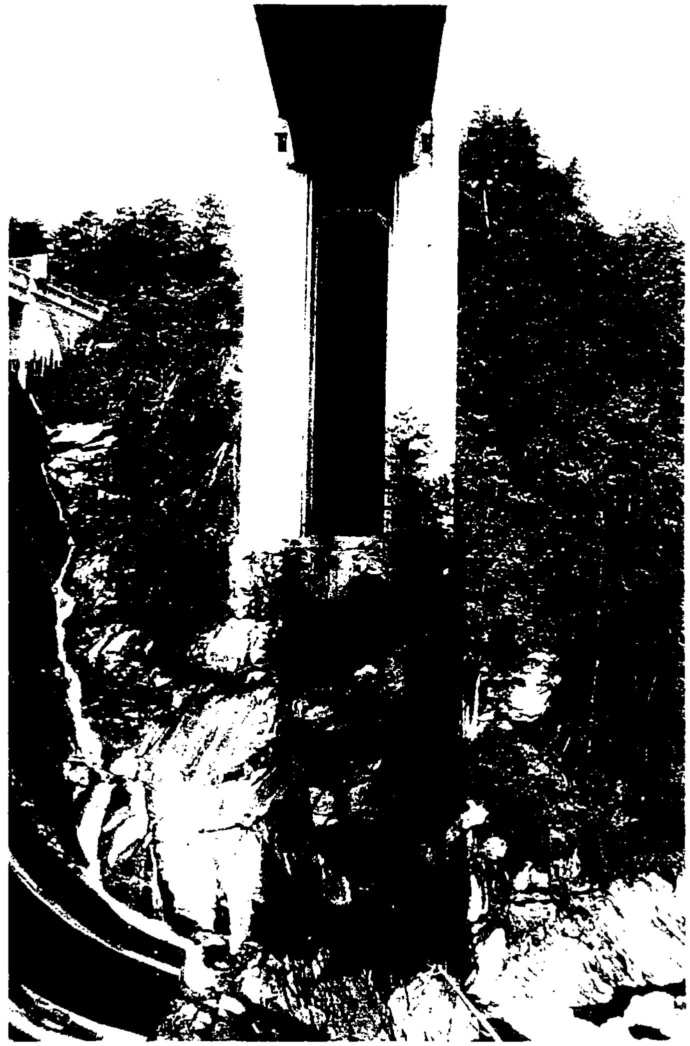


PHOTO 3 - PIER 2

PHOTO 4 - DAM CREST WITH FLASHBOARDS

in order to assess overall rock mass stability under existing and proposed loads. Effects of water flow through and over the rocks were also observed. In general the slopes appeared to be well drained with open foliation and joints providing preferred paths for the flow of groundwater.

GEOLOGIC MAPPING

The nature of the study required that care be taken to locate existing structures and observed geologic features with sufficient accuracy to allow rigorous analysis of the relationship of these features under various loading conditions. Steep slopes and difficult terrain necessitated the use of rapelling equipment for surveying and geologic mapping (Photos 5 & 6).

Horizontal mapping lines were located along the slopes of the gorge. The ends of the mapping lines and abrupt changes in the line direction were surveyed. On moderate slopes, the mapping lines were spaced 5 feet apart. Ten-foot spacings between mapping lines were used in areas of steep to vertical slopes. Mapping lines in the floor of the gorge were spaced on 10 feet centers.

Discontinuities at the site were located and described by systematic mapping. Field data collection activities were designed to collect a representative sampling of all significant discontinuities and major individual discontinuities. Each mapping line located by survey was used as a traverse line to identify the discontinuities for description. Features were located by their intersection with the traverse line. Features that did not intersect the traverse lines were located by reference to points on the lines above or below the feature.

Data were systematically collected for each discontinuity observed and included:

Rock Type and Hardness - The hardness scale used considered the rock's reaction from being struck with a geologic pick as well as degree of weathering.

Type of Discontinuity - Typically fractures, joints, faults, foliation, mineralized zones and pegmatite veins.

Strike and Dip - Individual exposed planes were measured, continuity and degree of waviness determined how many observations were required to adequately describe the feature.

Width and Spacing of Joint Sets/Fracture Zones - Discontinuities occurring as zones were recorded as such.

Length and Continuity of Feature - Continuity of the feature was evaluated based on the amount of intact rock observed along the discontinuity and expressed as a percentage (e.g. zero, zero to 5% and greater than 5%)

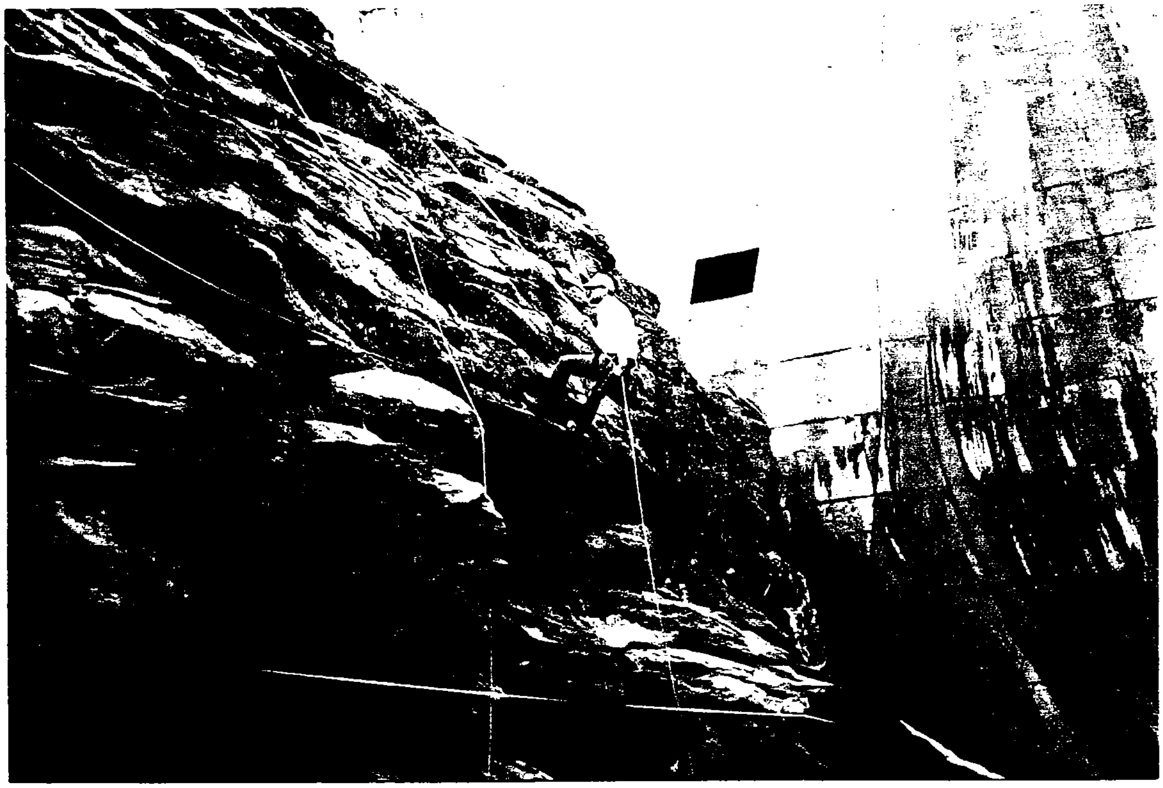


PHOTO 5 - SURVEY AND GEOLOGICAL MAPPING, PIER 1 SIDE

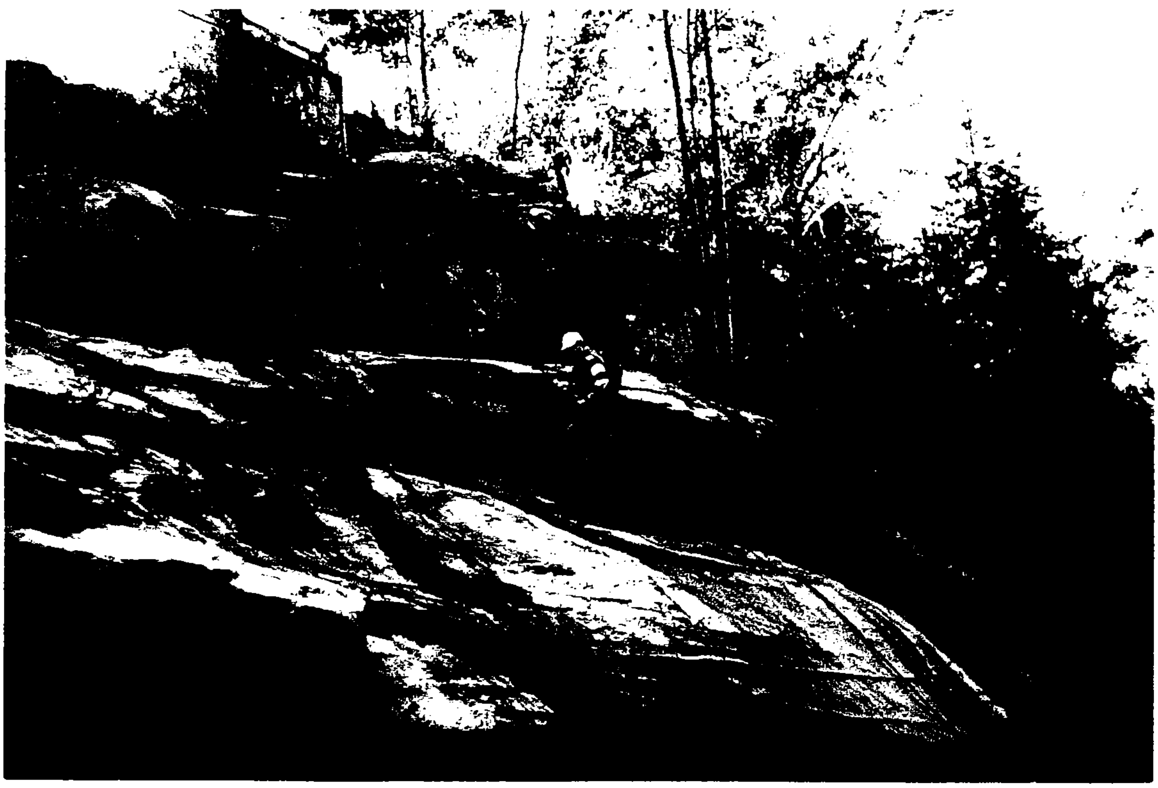


PHOTO 6 - SURVEY AND GEOLOGICAL MAPPING, PIER 2 SIDE

Roughness of the Discontinuity Surface - Typical descriptions included: slickensided, smooth, ridged, stepped and rough.

Type, Thickness and Hardness of Fillings

Water Conditions- Descriptions included: dry, stained, damp, seeping and continuously flowing.

All discontinuity data collected in the field were entered into a computer database for data reduction and analysis. Numerical values were assigned to a specific observed condition within one of the categories described above. This permitted statistical evaluation and manipulation of data from all fields.

Over 500 discontinuities were described at the site. One high-angle fault was observed that appeared to cross the bottom of the gorge and terminate at the base of the Pier 1. Otherwise, most joints and fractures were observed to occur parallel to the direction of foliation at high angles and with tight surfaces. Slickensides were observed on a few surfaces, however, typically surfaces were rough. Most of the open discontinuities were observed on the slopes of both sides of the gorge near the foundation of the dam. A few pegmatite veins and other mineralized fractures were observed.

Most discontinuities were dry. Water was observed seeping from foliation planes, primarily on the northwest side of the gorge in the vicinity of Pier 2.

GEOTECHNICAL INVESTIGATIONS

Exploratory Drilling

Because of difficult access to the pier foundation, a portable lightweight drill rig had to be used for exploratory drilling. Two BX size boreholes, one in the vicinity of each pier foundation, were drilled inclined 20 degrees off vertical and oriented into the slopes in order to maximize the number of discontinuities penetrated. The borehole near Pier 1 foundation was drilled to a depth of 18.7 feet, with core recoveries from 75 to 100 percent and RQD (Rock Quality Designation) ranging from 22 to 90 percent. The borehole near Pier 2 foundation, was drilled to a depth of 28.3 feet. Core recoveries in this boring ranged from 70 to 100 percent and RQD's ranged from zero to 100 percent. In both borings, the rock encountered was classified as hard to very hard quartzite with occasional thin (1/4 to 3/4 inch) lenses of graphitic schist. Slight weathering of the rock, indicated by staining of discontinuities, was confined to upper eight to ten feet of depth. Due to lightweight construction of the drilling equipment and hardness of rock, the drilling rates were generally less than one foot per hour.

Laboratory Testing of Rock Cores

Direct shear tests were performed on seven rock core samples using a specially designed shear testing apparatus (Photo 7). The testing apparatus was built to accommodate BX size cores and discontinuity orientations ranging from perpendicular to nearly parallel to the core axis. The cores were mounted in four inch diameter steel tubes, using capping compound. The discontinuity plane to be tested was fixed in such a manner that the surfaces could be aligned and matched prior to testing.

A pneumatic cylinder was used to apply a constant normal load perpendicular to the plane. An instream Universal Testing Machine was used to apply the vertical or shearing load. An initial normal load of 100 pounds was used. After the planes had translated approximately 0.3 to 0.4 inches, the samples were realigned to the pretest position and the test repeated at higher normal loads of up to 600 pounds.

The results of all the direct shear tests are presented (Figure 2). All the test data falls within a band, representing the ranges of a Mohr strength envelope. The slope of the envelope at any normal stress level represents the phi (ϕ) parameter, which is analogous to the friction angle of the rock discontinuity at that stress level.

Unconfined compression strengths of two rock cores tested were 7600 psi and 8100 psi.

ANALYSES OF FOUNDATIONS

Rock Mass Characterization

A comprehensive computer program package "ROCKPACK™ " by C.F. Watts & Associates was used for data reduction and analyses of the discontinuity data collected in the field, producing rectangular and stereonet plots of selected discontinuities, and identification and analyses of potential plane and wedge failures on the rock slopes.

Rectangular plots of dip vectors and stereonet plots of poles to the discontinuity surfaces were generated for the entire site and separately for zones in the vicinity of Pier 1 and Pier 2 foundations. From these plots, contoured stereonets of discontinuity density were then produced.

An analysis of all the discontinuity data indicated three major sets of discontinuities for the site. These three sets correspond to Joint Sets at N30E, N45W 85-90 and foliation at N30E 20SE. Data from the Pier 1 area below the existing foundation indicate all three sets of discontinuities. Data from the Pier 2 foundation area show the foliation and the northwest-striking joints but little evidence of the northeast-striking joints.

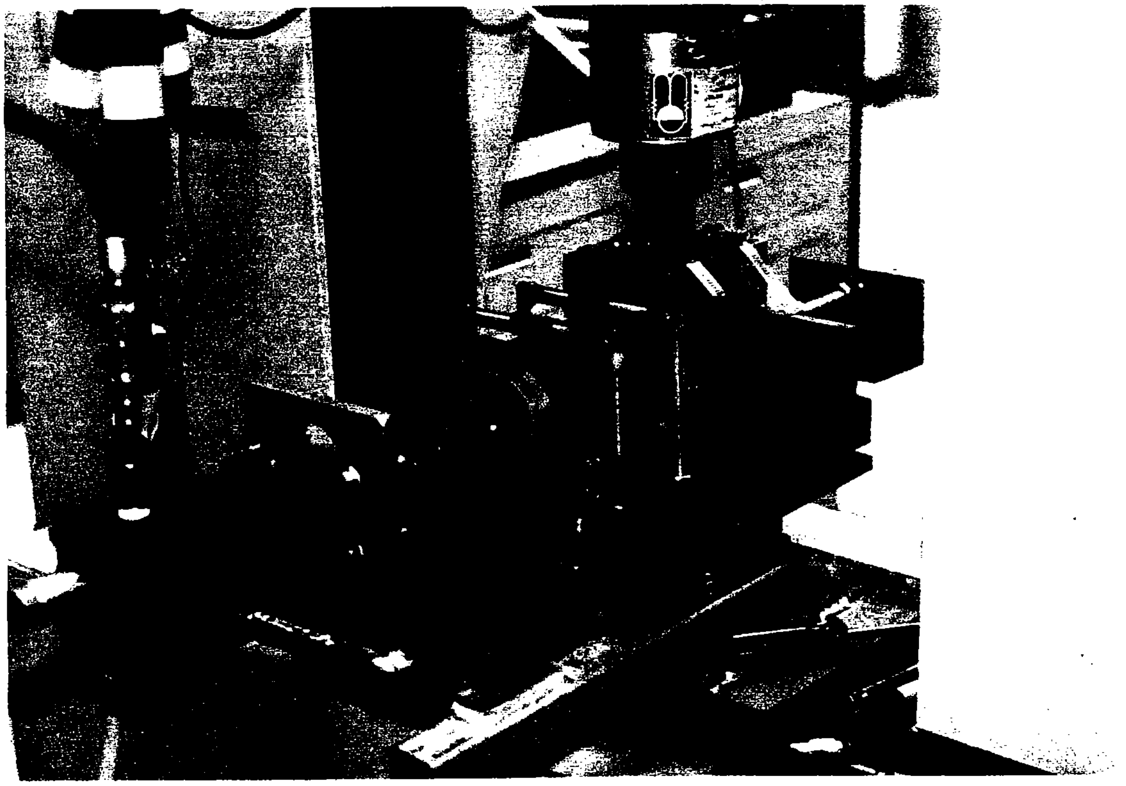


PHOTO 7 - SPECIALLY DESIGNED DIRECT SHEAR TEST EQUIPMENT

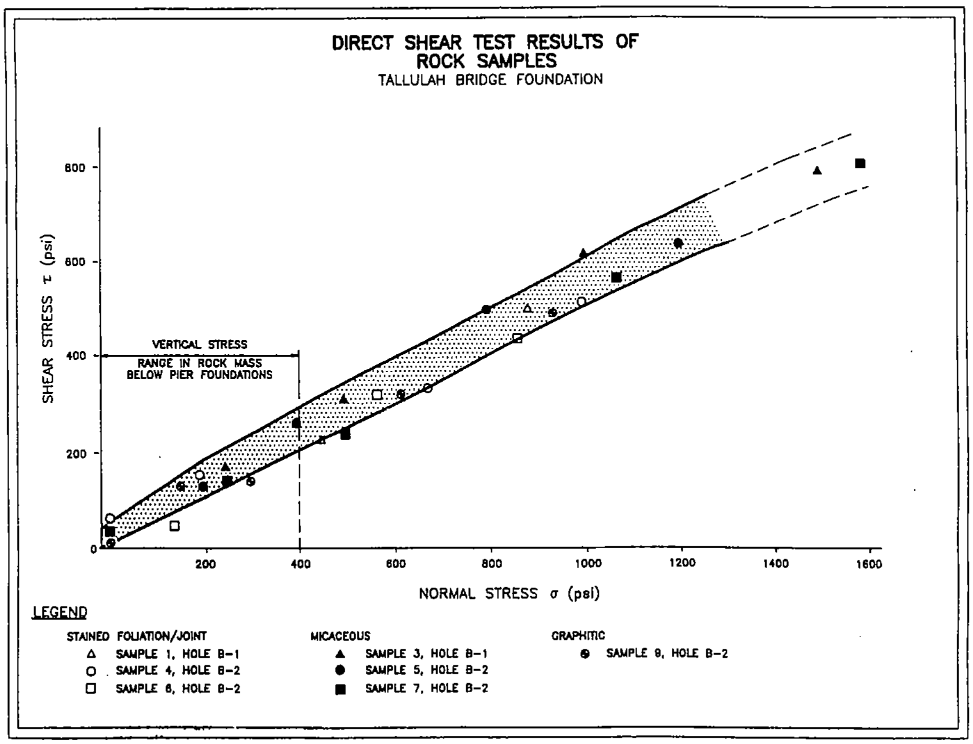


FIGURE 2

SELECTION OF POTENTIAL FAILURE PLANES AND WEDGES

Based on site data potential surfaces in the rock mass could be plane or wedge sliding failures. This potential was initially identified by using MRKLND and GRTCRL Programs which are part of the ROCKPACK package. MRKLND plots a great circle projection of the slope face and a circle representing the friction angle of the rock on a stereonet of discontinuity dip vectors. GRTCRL creates stereonet plots of great circles indicating statistically representative clusters of discontinuities and individually significant discontinuities with a projection of the slope face and a circle representing the friction angle of the rock mass. The great circles were plotted using statistically representative orientation data for discontinuities and minimum friction angle parameters determined from direct shear testing of rock cores. A $\phi = 22°$ was considered appropriate for plotting friction circles. Plots for areas below Pier 1 and Pier 2 foundations are presented in Figures 3 through 6. The shaded area between slope face and the friction circle is the "critical zone". Only those dip rectors and discontinuity intersections that fall within this "critical zone" represent the kinematically possible plane and wedge failures, which need to be considered for stability analyses.

Based on this kinematic analysis, only one potentially capable failure plane and one potential failure wedge was identified in the rock mass below Pier 1 foundation. On the Pier 2 side, six potential failure planes and three potential failure wedges were identified in the rock mass below the pier foundation. The effects of loads transferred to potentially capable failure surfaces are highly dependent on the location of these surfaces relative to the Pier 1 and Pier 2 foundations. To confirm the locations of selected potential failure planes and wedges relative to the foundations, details on photographs of Pier 1 and Pier 2 areas were compared with the statistical analysis of the data. Locations of the potential failure planes were then confirmed in the field. Below Pier 2 foundation, only one potentially capable wedge exists, located below the southwest corner of the foundation. Below Pier 1 foundation a potential wedge was determined not to be a problem after analysis of its geometry relative to the bottom of the gorge. In both cases full stability analyses were performed for these wedges. They were determined to be stable.

FOUNDATION LOADS

Various load groups were considered for evaluation of foundation stability. Load Group II, which in this case is dead load plus wind load, was considered as critical for analysis. Foundation pressures acting on the rock mass underlying pier foundations were computed using the service loads for existing conditions, Stage I widening and the final condition after widening of the bridge. For purposes of analysis, maximum possible foundation loads were calculated considering a rock surface 20 feet long (full width of pier foundation) and 10 feet wide along the length of the pier foundations. Average maximum loads acting on the rock mass per foot width of the slope were then computed for use in the stability analyses. These loads ranged from 142 to 270 kips per foot width of rock slope.

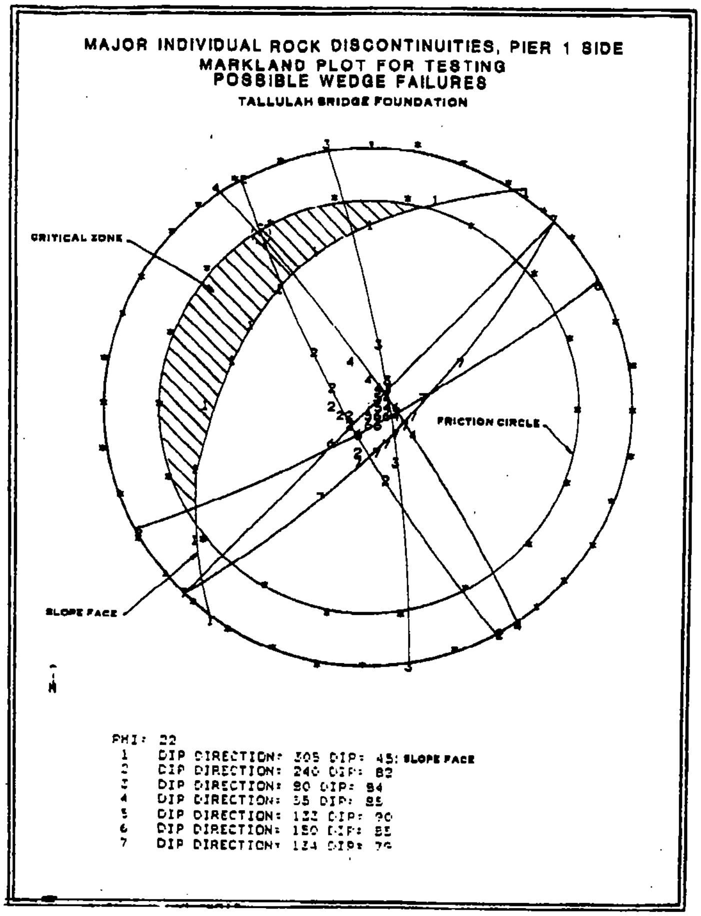


FIGURE 3

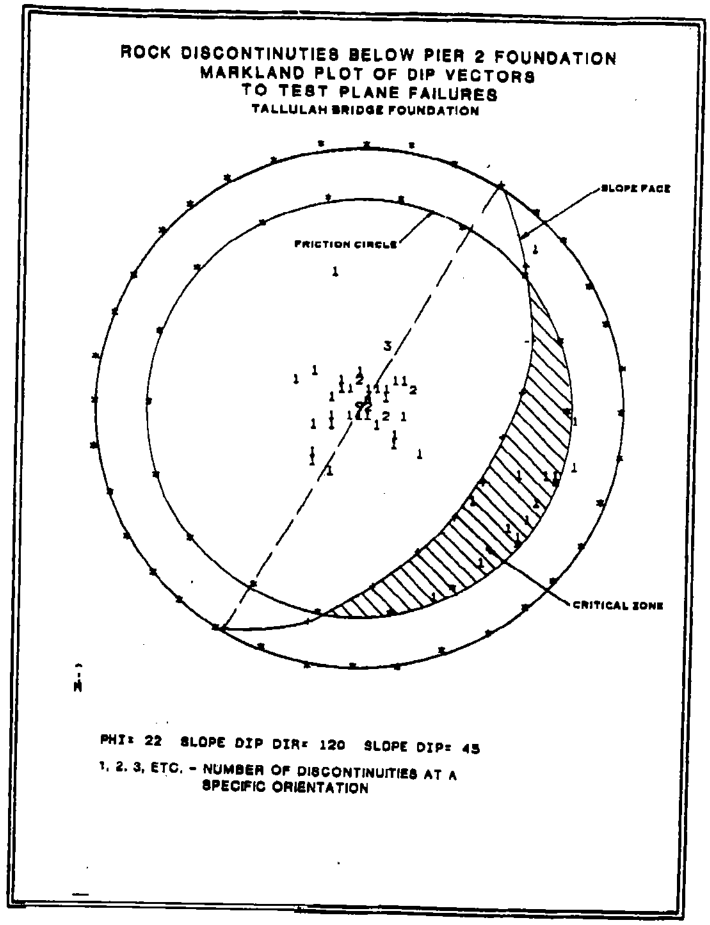


FIGURE 4

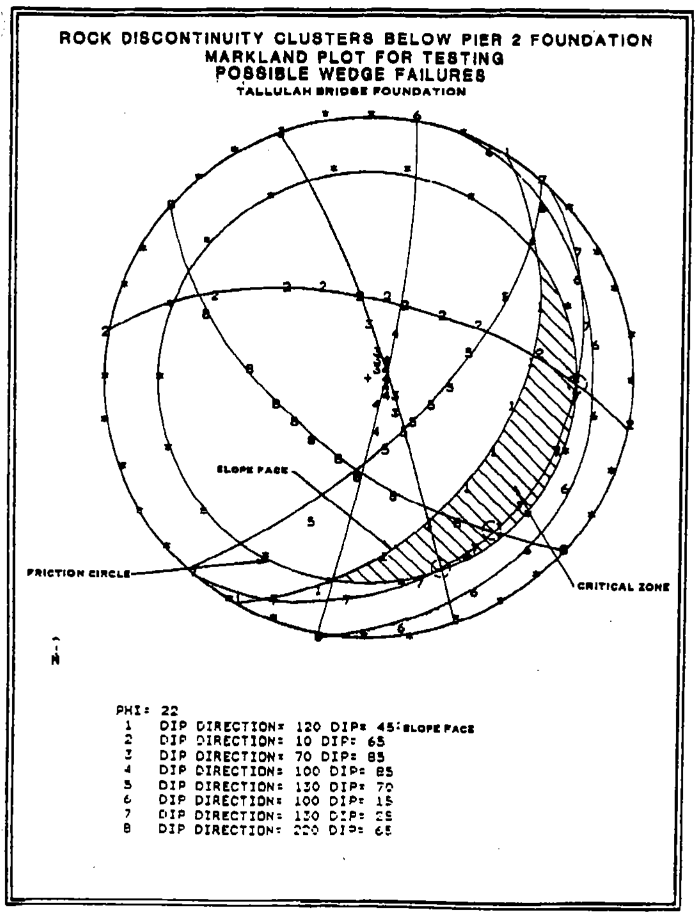


FIGURE 5

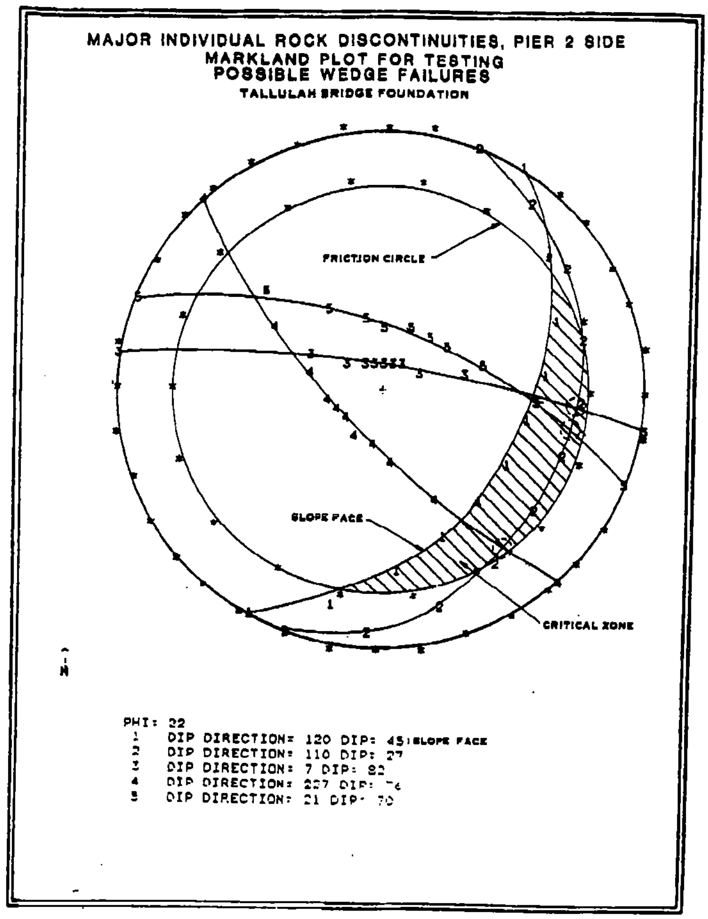


FIGURE 6

Parameters Used and Other Assumptions

Pier 1 side slopes were assumed to be dry (zero percent saturation). However, on Pier 2 side, where seepage of water was observed through discontinuities, a saturation value of 0.5 was considered. Because of potential for freezing temperatures and formation of ice which could restrict the flow of water for some time, a saturation of 1.0 was considered for short term conditions with a reduced factor of safety requirement.

The bridge site is located in seismic zone three, for which effective peak horizontal acceleration is 0.1 g with the return period of 475 years. Accordingly, maximum earthquake forces equivalent to 0.1 g were considered.

The following minimum factors of safety were considered for stability evaluations:

50 percent saturation or dry with earthquake loads: 1.25
100 percent saturation with no earthquake load: 1.25
100 percent saturation with earthquake load: 1.1

Based on results from direct shear tests, back analysis of an existing potential failure plane, and published data, it was considered appropriate to use values of cohesion (c) of 700 psf and friction angle, $\phi = 28°$ in the stability analyses.

STABILITY ANALYSES

PLANE and CMPWEDG Program, which are part of the ROCKPACK package were used to conduct stability analyses of planes and wedges respectively. The solutions are based on limit equilibrium theory (Hoek and Bray, 1981). Another program, WEDGE, developed by E. Hoek for wedge failure analysis, was used to check the results of CMPWEDG.

The results of the failure plane analyses indicated that the stability of rock mass below Pier 1 foundation is not endangered due to widening of the bridge. However, suitable anchoring of a few localized planes (particularly those dipping towards the gorge) was recommended to minimize any uncertainties associated about the extent and structural character of these planes.

For the rock mass underlying Pier 2 foundation, the factor of safety for an assumed 50 percent saturation and earthquake force ranged from 1.2 to 1.46 for different failure planes. For the most critical plane, the factors of safety corresponding to 100 percent saturation with earthquake load were computed as 1.08 and 1.03 for existing and future loads respectively. Even though such a transient loading condition is considered to be an extreme situation, provision for such an eventuality needed to be made to ensure long term stability of the rock mass below Pier 2 foundation. Systematic bolting was, therefore, recommended for Pier 2 side rock mass.

The results of the analyses on potential failure wedges near Pier 1 and Pier 2 indicated factors of safety ranging from greater than 2 to 12.

DESIGN OF ROCK REINFORCEMENT

For the rock mass on Pier 1 side, systematic rock bolting was not considered necessary. However, some local pattern or spot rock bolting was provided to tie blocks of rock bounded by the fault plane and a major joint. Dowels were provided to secure smaller blocks. The rock bolts are No. 11 high strength steel anchors, 50 feet long, tensioned to 35 kips. The vertical dowels are No. 14 steel bars, 15 feet long.

A systematic rock anchoring system was designed for rock mass below Pier 2 foundation. The PLANE program was used in design. The rock bolting system was designed to ensure adequate factor of safety for existing and future loads. Earthquake and seepage forces were also considered. Five rows of rock bolts of No. 11 high strength bars 20 to 30 feet deep and inclined at about 45 degrees were provided. These bolts were tensioned to 35 kips. Nearly horizontal dowels of No. 14 steel bars, 10 feet long. were provided in the rock mass above and behind the pier foundation to keep the small rock blocks in position.

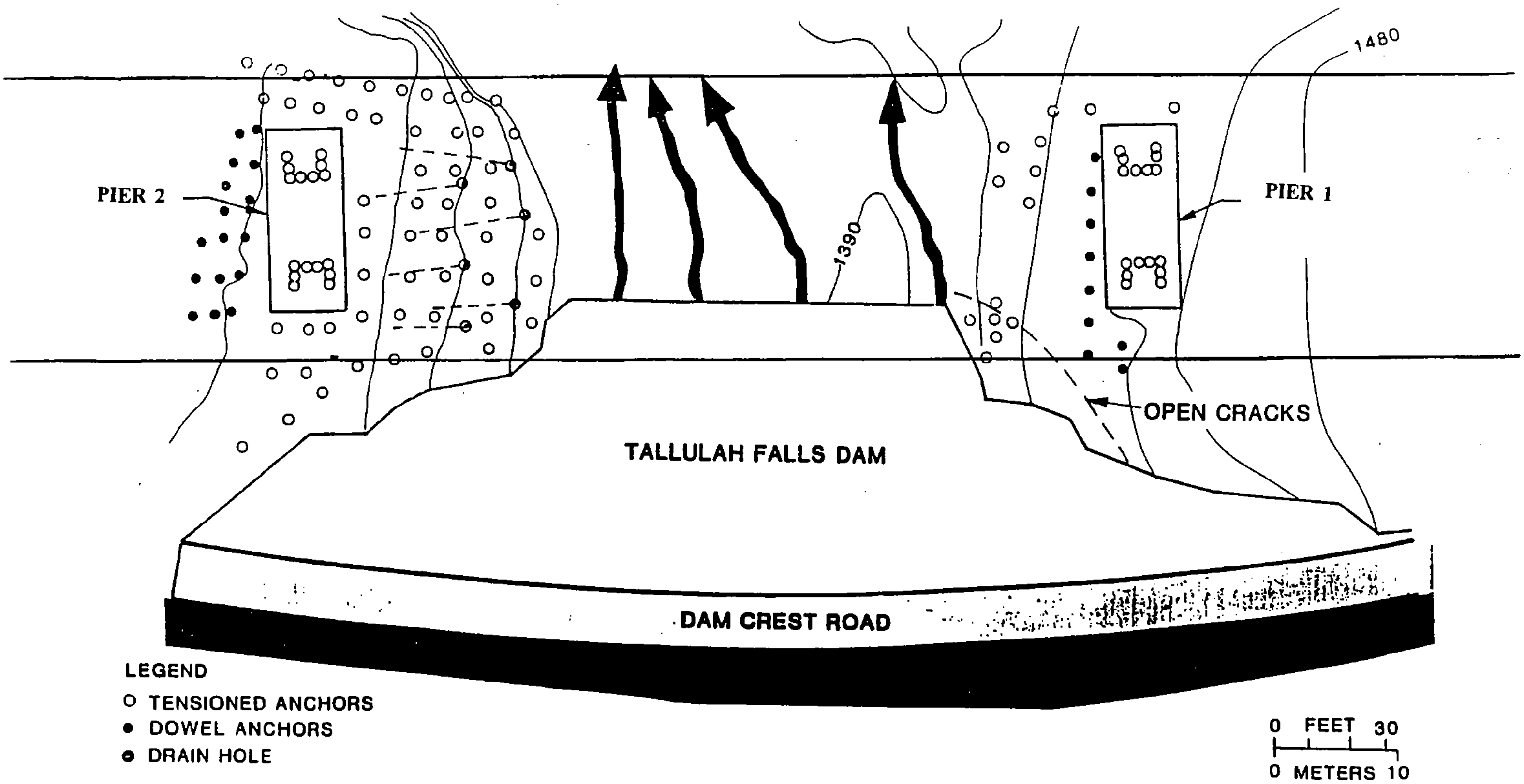
The rock bolting system and dowels for both Pier 1 and Pier 2 sides are shown on Figures 7 and 8.

Because some seepage of water was observed from foliations in the rock mass below Pier 2 foundation, sub-horizontal to horizontal drain holes 20 feet deep were installed.

Design plans called for strengthening of existing pier towers. New columns, 4.5 feet x 4.5 feet were constructed inside each pier tower. Six tensioned anchor tendons consisting of high strength steel strands, were used to tie these columns into the foundation rock for both pier foundations. These anchor tendons were installed from within the pier foundations and were of approximately 50 feet and 60 feet long for Pier 1 and Pier 2, respectively. The lower 25 feet of the tendons were grouted into the rock below and each anchor was tensioned to 228 kips. Plastic sheathing was used along the free length for corrosion protection and the space inside and outside the sheath was grouted after tensioning.

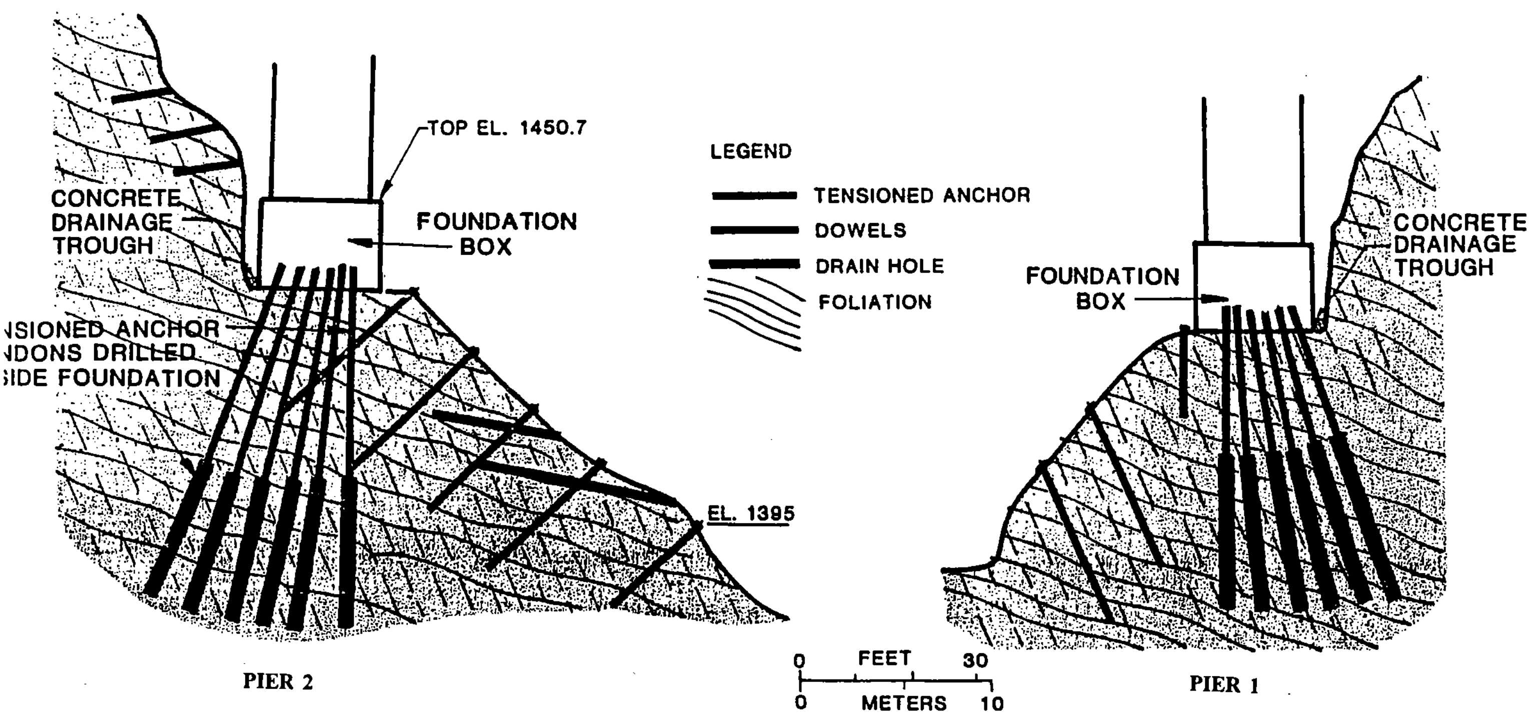
CONCLUSIONS

The results of the study and analyses indicate that the rock mass below the pier foundations is stable for existing loading conditions. However, for additional loads due to widening of the bridge combined with earthquake and seepage forces, systematic rock anchoring of the rock mass below Pier 2 foundation was necessary. Localized rock anchoring on Pier 1 side was provided to anchor major rock blocks and provide some measure of additional safety.



TALLULAH GORGE BRIDGE AND TALLULAH FALLS DAM

FIGURE 7
(ADAPTED FROM SOWERS, 1993)



TOWER ANCHOR AND DOWELS: SECTION

FIGURE 8
(ADAPTED FROM SOWERS, 1993)

ACKNOWLEDGEMENTS

The program for Tallulah Bridge foundation rock mass analysis and design of rock reinforcement system was commissioned by the Department of Transportation, state of Georgia.

The authors would like to acknowledge the assistance and cooperation received during the course of this work from Mr. Paul V. Liles, Jr., Mr. Bob Mustin and Mr. Warren Bailey of Georgia Department of Transportation. We are thankful to Mr. Mustin and Mr. Bailey for their visit to the site and important discussion during our field work.

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ROCK FALL MITIGATION PROGRAM AT NANTAHALA DAM USING WIRE ROPE NETS IN CONJUNCTION WITH OTHER TECHNIQUES

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ABSTRACT

The rock fall mitigation program implemented at the Nantahala Dam was in response to a large rock slide that occurred in December 1993. The slide placed rocks in the dam's spillway but did not significantly reduce its' capacity. The owner subsequently implemented the mitigation effort on the area directly above the spillway structure and areas on both sides of the structure. The mitigation effort emphasized the use of several techniques to control the rock. This included rock bolts that varied in length from ten (10) feet to fifty (50) feet, chain-link mesh pinned to the cut and wire rope nets anchored to the cut with chain-link mesh. A rock slide into the spillway structure had the potential to damage it to the point where it was no longer functional and could cause a problem with downstream flooding. The successful completion of the work by the contractor in March 1995 gave the owner a system that greatly reduced the danger from a rock slide damaging the spillway structure and impairing its' safety.

INTRODUCTION

History Of Slope Protection

The concept of draping mesh over a slope to control loose rock has been used for several years and is still being used by several states. The mesh has been either chain-link fencing with two (2) inch mesh openings or twisted wire hexagonal mesh that has 3-1/4 inch by 4-1/2 inch openings or 2-1/2 inch by 3-1/4 inch openings. The mesh is either galvanized or PVC coated for corrosion resistance and esthetic purposes. Generally, the drape lies directly on the slope and anchors go across the top. Additional anchors could go in the mesh to hold it down and give it additional resistance to sliding or rolling rocks and debris. The purpose of the mesh has been to allow the rocks to move down the slope or cut in a controlled fashion and land in a small catch area at the base. Figure 1 is an example of a recent project undertaken by the Pennsylvania Department of Transportation using twisted wire hexagonal mesh as a drape on a slope with interior anchors.

Figure 1

This approach has the advantage of not needing a large catch area. The common application is along a highway where the shoulder is narrow too nonexistent and laying back the slope is not economically viable. This type of solution is also attractive from the perspective of being a passive system that does not require a great deal of maintenance.

The combination of wire rope nets and mesh for a slope drape produces a system with greater strength and the ability to control larger rocks and debris flows without being damaged. The system still functions in the same fashion but the wire rope net is now a structural net to support the smaller mesh. It has the ability to absorb the static and/or impacting energies associated with larger rocks and debris flows. In cases where the rocks are larger than two (2) feet in diameter or weigh in excess of 1,350 pounds, wire rope nets are more suitable for the cases instead of mesh. Ring nets have been used to retain extremely large rocks instead of the wire rope nets. Wire rope nets made from multi-strand wire rope have much greater strength than the single wire chain-link or hexagonal mesh material. The other approach is to place anchors within the mat and pin it to the slope to keep rocks and debris on the slope. The wire rope nets lend themselves to this type of application. They are made in panels and an anchor would go at the corners. The nets have been used with and without the smaller mesh to hold rock in place. Figure 2 is photograph taken in New Jersey where the nets were used to hold rock in place and a smaller mesh was not incorporated. In this situation, the rock is fairly solid and would break loose in large blocks. There was no real concern for smaller rock and debris that might break loose. The Nantahala Dam is the most recent application of pinning wire rope nets with a smaller mesh material to the slope to hold the rock in place.

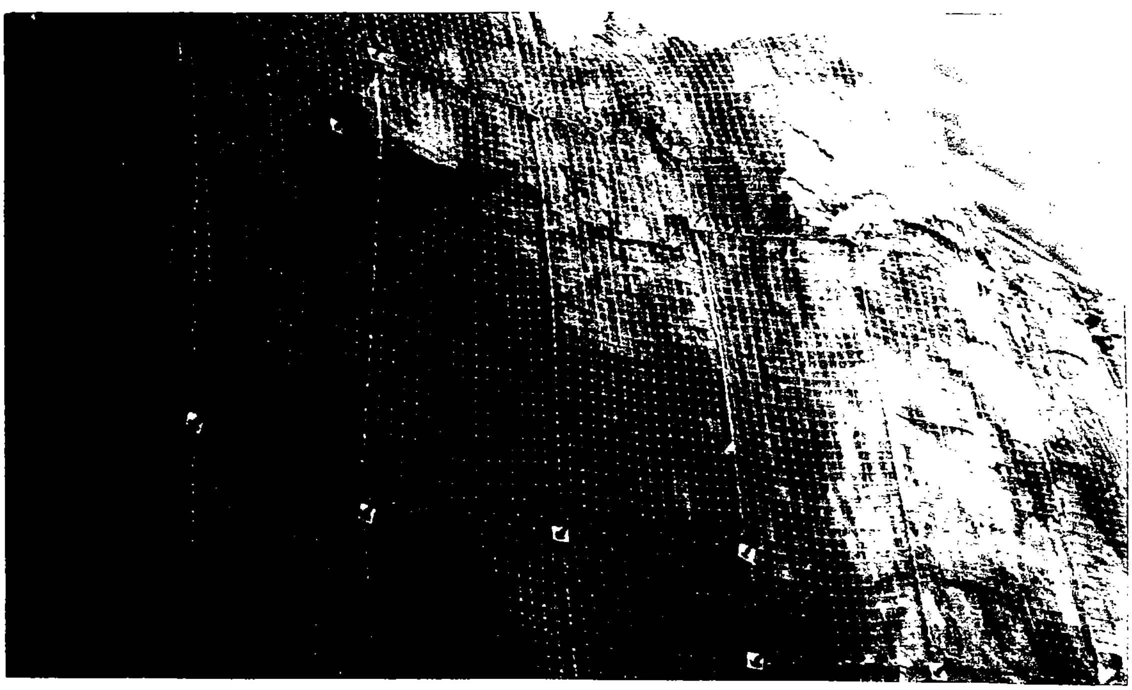
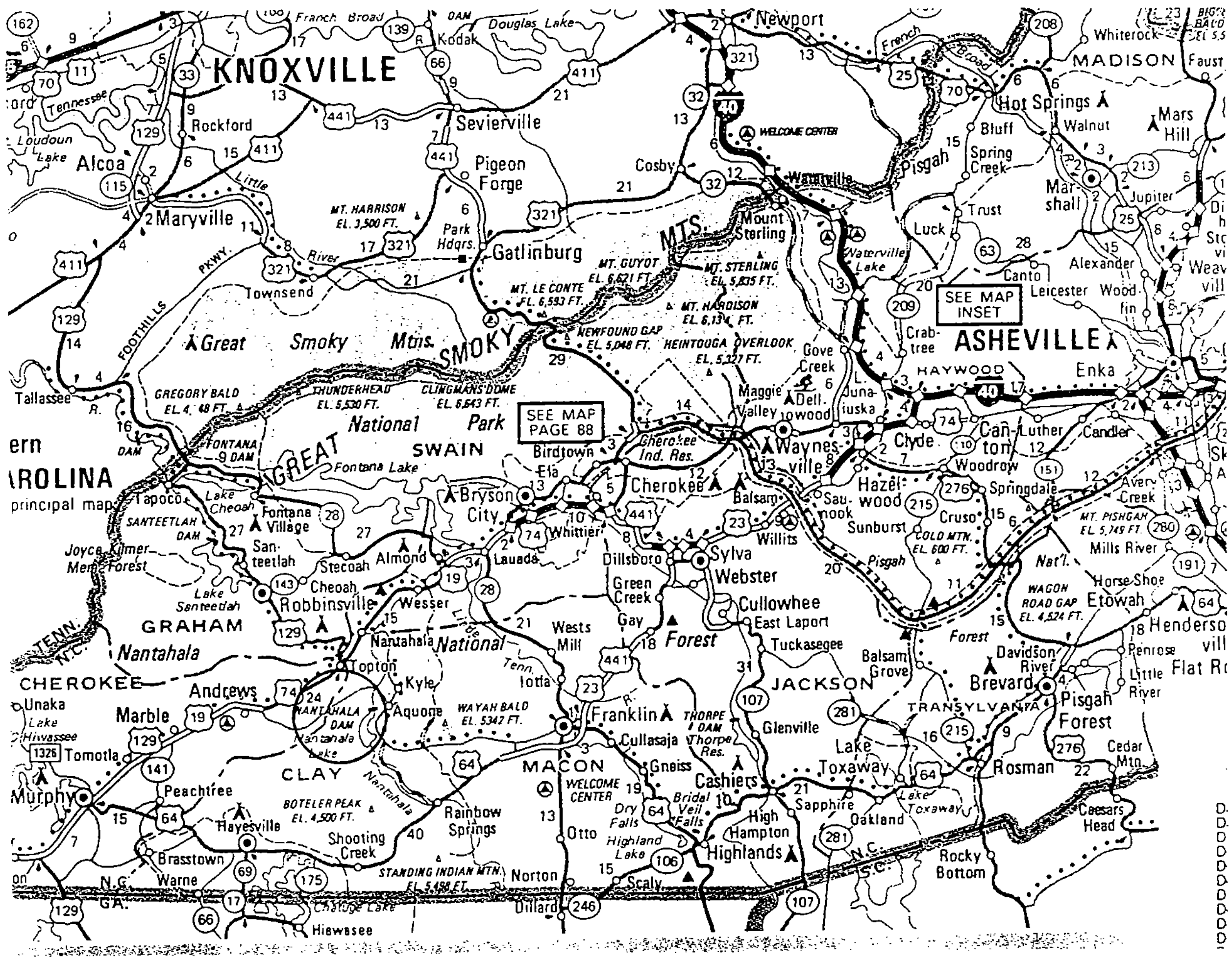


Figure 2

The Nantahala Dam is near Topton, North Carolina in the Nantahala National Forest. It is approximately a two (2) hour drive from Ashville through the beautiful Nantahala Gorge (Figure 3). The dam's current owner is the Nantahala Power and Light Company in Franklin, North Carolina. The company is a wholly owned subsidiary of Duke Power in Charlotte, North Carolina. The dam was built between 1942 and 1944 by Alcoa Aluminum to provide power to their factories in Tennessee. The hydro plant is several miles away from the dam and has an output of forty-four (44) megawatts.



<u>Figure 3</u>

The dam's spillway is on the East side and it was cut along one side of a mountain. The condition of the rock cut today is essentially the same as the one left after the dam was built. The cut has not changed much since the construction was completed over fifty (50) years ago. This was true until December 1993, when a large debris slide occurred and it went into the spillway (Figure 4).

Figure 4

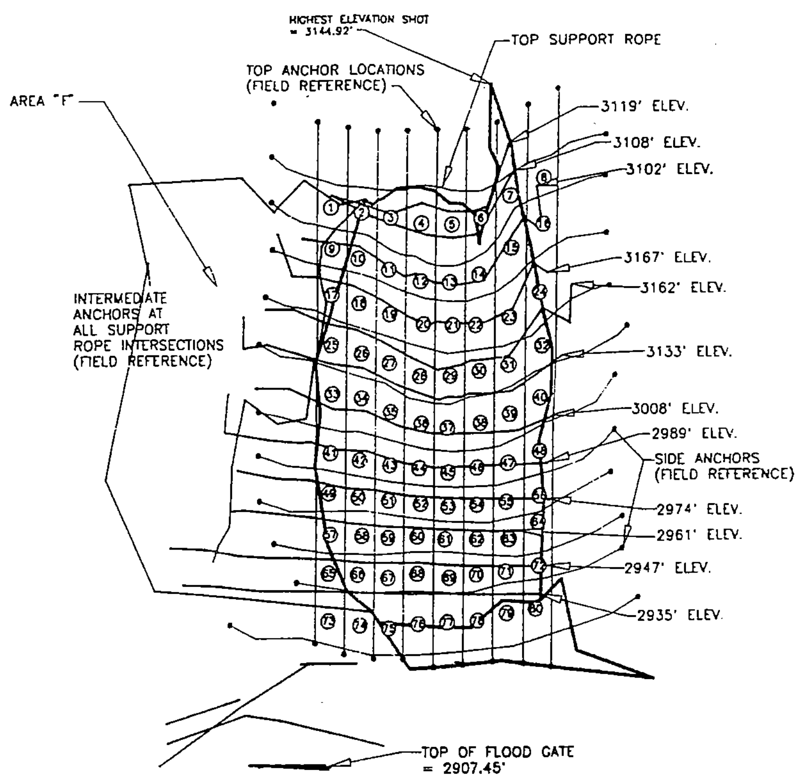
The material in the debris slide varied from small rocks to large blocks that were up to twenty (20) feet in length. The largest block was probably 4 feet by 8 feet by 20 feet in size. The estimated quantity of material that fell into the spillway was 6,000 cubic yards. The rock and debris went into the spillway and did not damage the spillway structure nor did it significantly impede the spillway's discharge capacity. Removal efforts did not start until Fall, 1994, when it was possible to drain the lake below the spillway for access. This allowed the clean up contractor to work in the spillway and clear out the debris by pushing it into the lake.

During Winter 1994, Duke Power began to look at the overall condition of the spillway cut through the mountain for future problems. Their geologist reviewed the slide and the cut and concluded another slide could occur at some time in the future. The area adjacent to the slide is a similar formation and it has the potential to fail in the same fashion. This presented a problem as a future slide in this location would go into the spillway structure and possibly damage it. If the spillway structure were to become inoperable, a potential safety hazard would exist and pose undue risk to people downstream from the dam. As a result, a decision was made to develop and implement a mitigation plan to prevent rock from falling into the structure. It was at this time that we were contacted to discuss use of our nets to hold the rock in place on a permanent basis.

SLOPE PROTECTION SYSTEM DESIGN AND SPECIFICATION

We began discussions with Duke Power in late Winter 1994/early Spring 1995 the concept of placing our wire rope nets on the cut to hold the rock in place. It was important to them to prevent rocks and debris from falling off the slope and into the spillway structure. This resulted in a decision to use chain link mesh in conjunction with wire rope nets to contain the rocks and debris. Further, Duke prepared an extensive rock bolting plan in an effort to pin the rocks in place and to prevent their movement. The main purpose of the slope protection system was to contain surface rocks that were not pinned. They were concerned these rocks could become dislodged at some time due to the freeze-thaw cycle and weathering.

The approach we recommended to Duke was to use our nets in 12'6" x 25'0" panels and place them on the cut in such a fashion as to match the contour as closely as possible. We included with our recommendation using a cable grid based on the net's size to support the nets and pull them into the cut. The chain link mesh was to go on top of the nets and the seaming of the nets would include the chain link. The final aspect of our design was to install anchors at the support rope intersection points. Heavy duty eyes were fastened to these anchors bolts. They allowed the horizontal and vertical support ropes to move freely and be pulled taut to hold the nets snug to the cut (Figure 5).



FRONT ELEVATION VIEW

Figure 5

Figure 6

All the drilling and installation work was performed from a work platform suspended from the crane. The contractor used two different size air traks with the smaller one being used to access the highest anchor bolt locations. The air compressor for the drill remained on the ground and the air hose ran from it up to the platform. Two (2) guide ropes were secured to the platform and their purpose was to position the platform while the crane swung it into place. The air tracks on the platform performed the drilling and it was necessary to secure the platform to the cut before drilling (Figure 7).

Our design specified one-half inch diameter anchors set with resin that have a pull out strength of 12,000 pounds and no post-tensioning. When Duke Power's geologist was incorporating our design recommendations into the final design, he contacted us about using the same anchors for a second purpose. He was planning a spot bolting program on the slope and wanted to use the anchors bolts for this application. We looked at the application and felt it was not a problem. The use of the anchors for spot bolting would not impact the functioning of the slope protection system. The final plans specified these anchors be set to a depth of thirty (30) feet outside the pattern bolting area and set to a depth of ten (10) feet within the pattern bolting area.

The bolting program at the dam called for spot bolts and pattern bolts. Within the area covered by the wire rope nets, the spot bolting consisted of the support rope anchor bolts and others. The pattern bolts were placed at the base of the slope to lock the toe in place and prevent movement. It was felt that restricting movement in this area would preclude movement in the cut above this elevation. The plans called for ninety (90) pattern bolts set to a fifty (50) foot depth and tensioned to 142 kips after installation. These bolts were to be the last items installed as part of the mitigation efforts.

INSTALLATION

The contractor started work at Nantahala Dam on October 12, 1995. The owner had a crucial completion date in the Spring that the contractor had to meet. In order to meet this date, their plan was to work two shifts of ten (10) hours each and six (6) days per week right through the winter. Towards the end of the project, the contractor went to two shifts of twelve (12) hours each. The only time the contractor did not work was when the weather became nasty and it was too dangerous to work.

Prior to the contractor's starting, the owner had implemented a scaling program, during Winter 1994, to remove as much debris and rock as possible. This left the contractor with an open bare slope and reduced the hazard from falling rock and debris. The access to the cut was from a Manitowoc 4100W crane that had 280 feet of boom with 20 feet of jib. The combination of boom and jib was just enough to reach the top of the cut to drill and set the anchors and nets. An earth bench was built in the spillway for the crane and it raised the crane an additional fifteen (15) feet (Figure 6).

The location of the dam is remote and access is from secondary and unpaved roads that go through the Nantahala National Forest. This did not allow the crane to be brought to the site in a semi-assembled condition. The crane was trucked to the site in pieces and it took eighteen (18) trucks to bring all the pieces to the site where it was assembled and load tested before use.

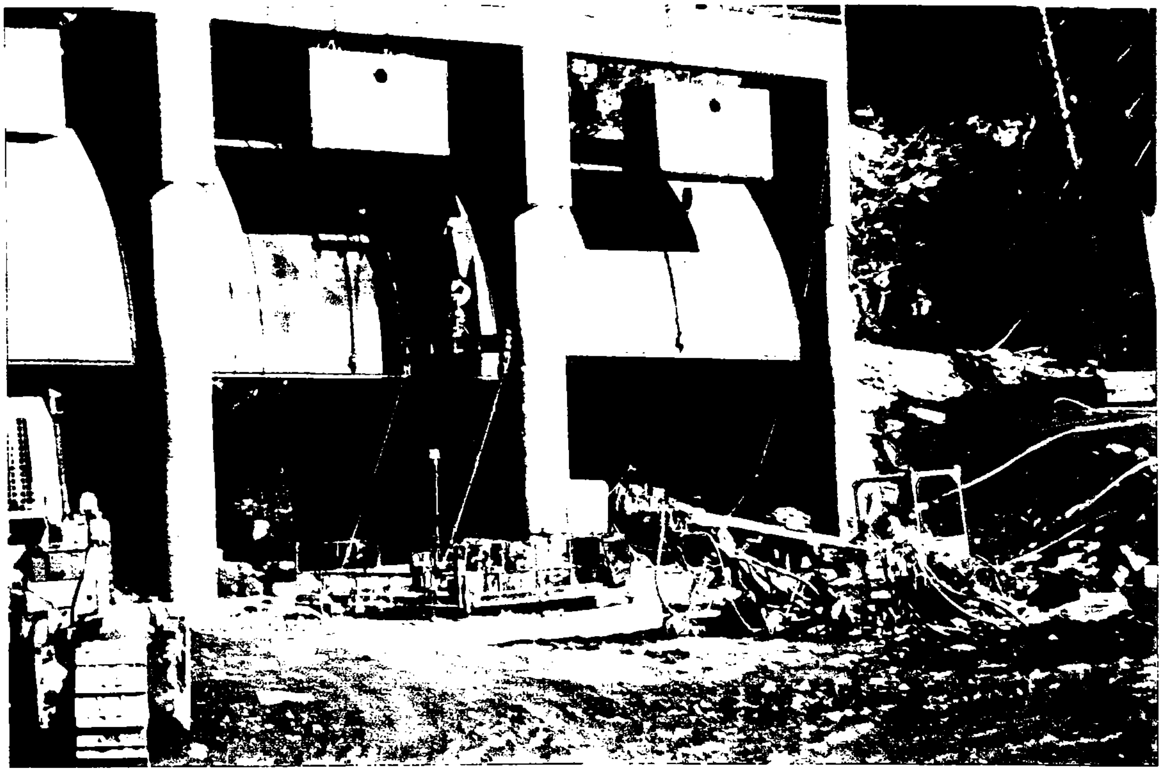


Figure 7

We supplied the contractor an installation procedure to help him understand the steps involved with the installation. The basic steps were layout system, install anchors, install support ropes, install wire rope nets, install chain-link mesh, seam mesh and nets together and to the support ropes. These were the basic steps we outlined in our procedure; however, several changes were made to the installation to suit field conditions. The geologist, owner and the contractor had a valid concern about the safety of the men on the platform and their exposure to falling rock. To address this concern, the chain link mesh was placed on the slope to act as a protective barrier and reduce exposure to falling rock. The other change made by the contractor was to have the support ropes lose to facilitate the seaming and he tightened the ropes after the seaming was finished.

After the chain-link mesh was installed, the contractor essentially followed the procedure. The cut is somewhat irregular and it was important to correctly locate the first horizontal support rope. We picked the elevation where the nets were to start and laid out the rope to follow the horizontal contour line. A vertical support rope was installed along the left side of the area and these two ropes became the reference lines. An anchor and vertical support rope were placed every 12'6" along the horizontal reference rope.

The key to the installation was the placement of the first horizontal support rope and the vertical ropes at 12'6" intervals along the reference horizontal rope. This took into consideration the shape of the cut and positioned the nets against it (Figure 8).

Figure 8

Once the vertical support ropes were in place, the contractor laid the first row of nets on the cut and tied three of the sides together. There was sufficient space on the platform to store six (6) folded nets and completely unfold one net before setting it in place. Our procedure said to use a rope with markings on it to locate the anchors and the horizontal support ropes. Instead, the upper nets were seamed together and to the vertical support ropes and the bottom of the nets was used as a guide for the next horizontal support rope and row of anchors. This approach worked quite well for the contractor and it gave him the true location of the nets on the cut and anchor locations. He had the latitude to move the anchors a foot along the ropes as necessary if it was not possible to place an anchor at the exact intersection location. As the contractor worked his way down the slope, he shortened the top rows and lengthened the bottom rows. He had to do this to address the widening of the cut area requiring coverage and the need to cover additional areas that were identified by the owner. The next to the last step in the installation was for the contractor to go back and install anchors in nets at selected locations.

The idea was to set an anchor in a net where the net went over an open area and did not lie on the slope. A cover plate on the anchor was to pull the net into the cut as much as possible and reduce the fall distance for any rock that might dislodge. The last step for the contractor was to pull the support ropes taut to hold them tight to the slope and to prevent loose rock from getting past them.

All the work was completed from the drilling platform and inspectors worked along side the work force. The owner had full time inspectors on the project to check and monitor the work of the contractor. Prior to the start of the project, we wrote and gave to the owner an inspection procedure for their inspectors to use. The procedure gave enough information and detail to the inspectors so they were able to understand what was being done. With this knowledge, they were able to inspect the work and spot problem areas as they occurred or before they occurred. The contractor worked diligently on the project and was able to complete the installation in Mid-March. The speed at which the contractor could work was limited by the crane and the platform. They had the one crane on site and it could support a single platform.

CONCLUSION

The successful completion of the project in Mid-March allowed the owner to refill the dam back to its normal level. The combination of rock bolts, chain link mesh and wire rope nets will allow Nantahala Power And Light to operate the dam for many years with a reduced concern for rocks falling and damaging the spillway structure. Periodic inspection and maintenance of the system are necessary to ensure the desired level of protection is not degraded. The last time we inspected the site, we observed that a rock had dislodged and fallen into the netting. The rock came from a joint that the net did not contact. The rock weighed approximately two hundred-fifty (250) pounds and it fell approximately twenty-five (25) feet before coming to rest against the nets and a horizontal support rope. The nets appeared to be undamaged; however, it did split apart the chain link when it struck the mesh. The rock had a flat shape and a sharp or jagged edge on one side. The esthetics of the installation were not a concern and galvanized chain link mesh and nets were installed on the slope. They do have a somewhat shiny look when seen up close but, from a distance they blend into the surroundings (Figure 9).

Since the inception of the company in 1985 in the United States, numerous slope protection systems similar to the one at the Nantahala Dam have been completed at our manufacturing plant in Santa Fe, New Mexico. The systems have been for users in the United States and world wide.

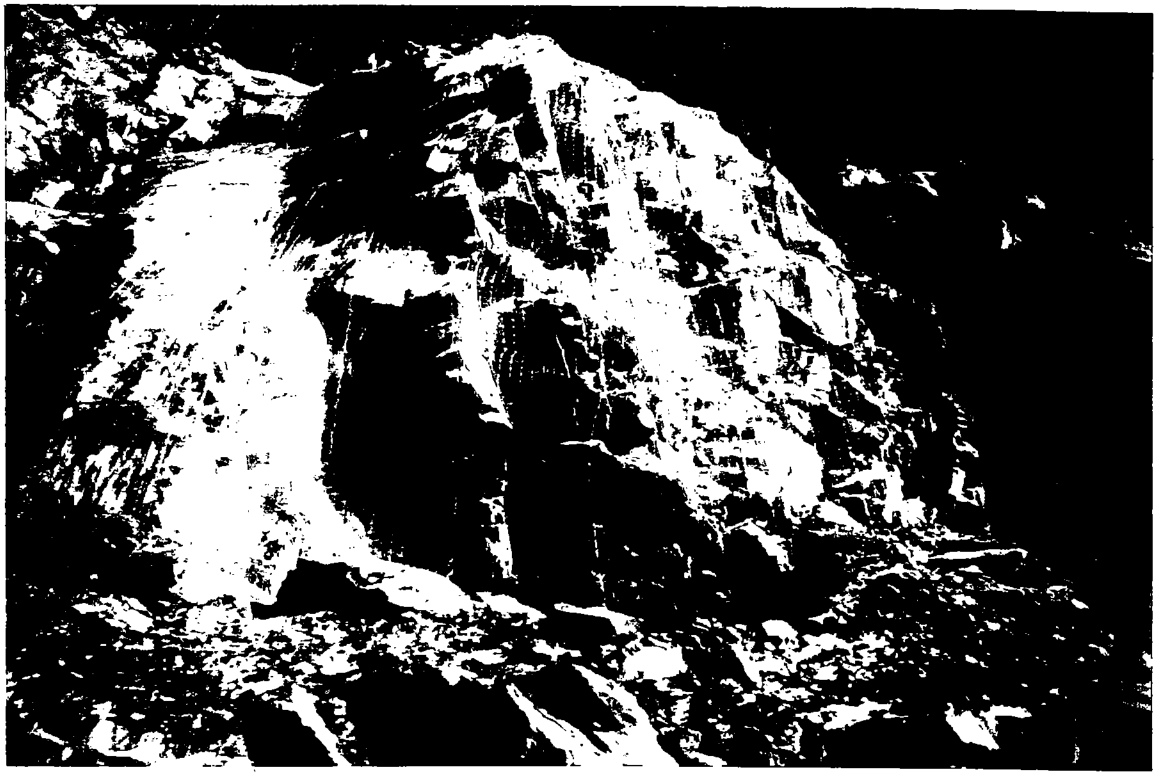


Figure 9

This paper was presented at the 46th Highway Geology Symposium, May 14 - 18, 1995, in Charleston, West Virginia.

HELICAL TIEBACK ANCHORS HELP RECONSTRUCT FAILED SHEET PILE WALL

by

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Walter Smith[2], Chance Foundation Specialist

ABSTRACT

Helical anchors have been used in various applications, including tiebacks in temporary and permanent earth retainment walls. In such applications, multiple helix anchor sections are power screwed or "torqued" into the soil. Upon installation, load is applied and transferred to the soil by the individual helices acting as bearing plates. This paper is a case history of a failed sheet pile wall along Route 29 in Campbell County, Virginia. A three hundred foot long sheet pile wall failed soon after construction due to heavy rainfall. The earth embankment behind the wall sloughed and the center section of the wall bowed out considerably. The Virginia Department of Transportation (VA DOT) selected helical earth anchors as tieback tendons for the wall repair. Helicals were chosen because their installed cost was 40% less than conventional tiebacks. The helical anchors were installed with an average free length of 25 feet above the top-most helix. Installation torques ranged from 7000 to 10,000 ft.-lbs. Construction was sequenced in stages of 50 feet. The original wall was pulled after the new wall was driven behind it. Tiebacks were then installed through and connected to the new wall. This process was repeated until the entire wall was complete. Performance or proof tests were conducted on all 34 anchors installed. Maximum test load was 32 tons. Design load was 24.5 tons, with a lock-off load of 18 tons. Results show that a sheet pile wall using helical anchors as tiebacks is a feasible, economic method of construction of permanent earth retainment walls.

PROJECT HISTORY

The original sheet pile wall was installed to retain a steep sloped embankment along Route 29 in Campbell County, Virginia near State Route 699 and the town of Altavista. This wall was installed with a batter toward the embankment. No tiebacks were originally used. Due to heavy rainfall soon after construction, the earth embankment behind the wall experienced a slope failure and sloughed toward the wall. As a result, the wall bowed out in the center one third of its length and rotated away from the embankment and toward the highway. The primary reason for the failure was determined to be the lack of adequate drainage from behind the sheet pile wall. The increased load contributed to the wall rotation and slope failure.

These events prompted the VA DOT to issue project number SLR-313-94 to reconstruct the failed sheet pile wall. The project was opened for bids on August 24, 1994. Tieback installation did not begin until December 13th of the same year, and was not complete until January 12th, 1995. Of that time period, twelve working days were used to install, test, and lock off the helical tieback anchors.

ANCHOR SELECTION

Both conventional grouted anchors and helical anchors were bid as tiebacks for this project. Helical anchors were chosen because their installed cost was forty percent less than conventional tiebacks. There were several added benefits derived. One was that helical anchors do not require grouting, thus the problems of handling water and grout mix in cold weather were eliminated. Temperatures during installation of the tiebacks ranged from 15° to 65° F. Also, no special equipment was needed. The general contractor for the project was Wilkins Construction from Amherst, Virginia. J. A. Walder, Inc. from Ashland, Virginia, was the sub-contractor who installed the helical tiebacks. The sub-contractor was able to adapt and use the general contractor's backhoe. This saved on mobilization and demobilization costs.

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Accessibility was a problem that made helical tiebacks attractive. At its closest point, the wall was about twenty feet from the highway. In addition, the tiebacks usually had to be installed about 10 feet above the highway with a slope from the wall to the ditch below the highway. This situation would have made it difficult to drill for grouted anchors. The construction scheme for grouted anchors required suspending a drill next to the sheet pile wall with a crane. The construction scheme for helical tiebacks, as described later, did not require this.

HELICAL TIEBACK ANCHOR CONFIGURATION

Soil boring data was used to determine the helical anchor configuration. Unfortunately, only one soil boring was conducted on the hill above the sheet pile wall. Thus, the information from only one soil boring was used to assess the strength parameters of the soil behind the wall.

The soil profile as reported in soil boring one consisted of a red clay near the surface, changing to a micaceous silty clay, then silt, and finally micaceous sand before reaching bedrock. Bedrock consisted of micaceous schist. Soil boring one and a generalized overhead view of the job site are shown in Figure 1.

The required helical anchor type as determined from the soil strength properties from the boring was a trade name "SS175". The SS175 consisted of a solid steel 1-3/4" round-cornered-square (RCS) shaft with four helices. The helices were 8" - 10" - 12" - 14" diameter helically shaped bearing plates arranged on a 10 foot lead section. 1-3/4" RCS plain extension sections were used to extend the anchor to the proper depth behind and below the sheet pile wall. All helical anchor components, leads, extensions and Dywidag bar adapters were hot dip galvanized per ASTM A-153. The Dywidag bar was galvanized as supplied.

The ultimate tensile strength of SS175 helical anchors is 100,000 lbs. and the ultimate torsional capacity is 10,000 ft.-lbs.

CONSTRUCTION SEQUENCE

Construction began by driving new sheeting behind the failed existing sheeting. VA DOT directed the work to be done in stages of 50 ft. for a total length of three hundred feet. After new sheeting was in place, the old sheeting was pulled. Soil between the old and new sheeting was excavated, sloped and a small bench formed to allow a work area for tieback installation.

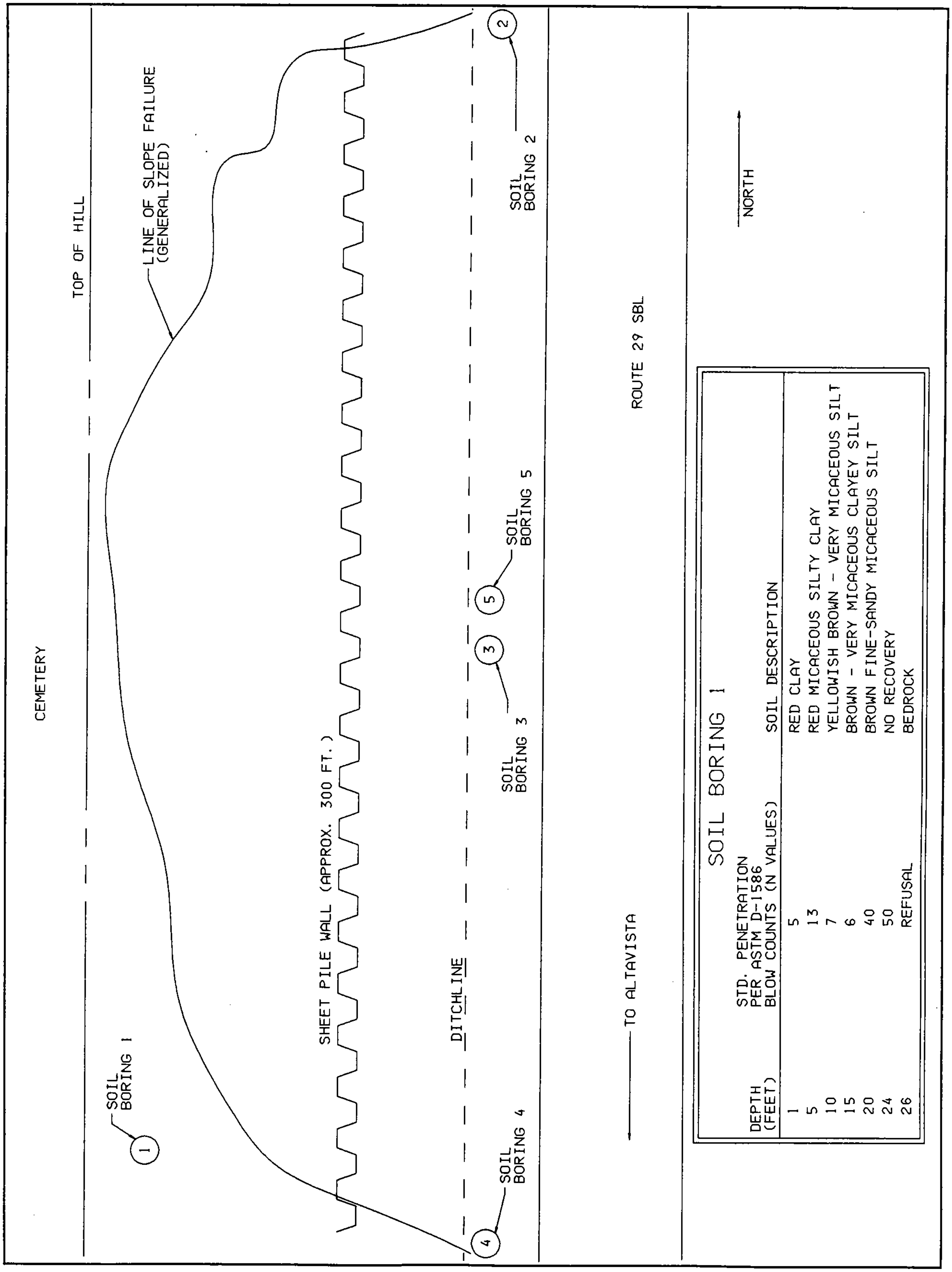
At the specified location below the top of the sheet pile wall, a torch was used to cut a fourteen inch diameter hole through a sheeting joint on the face of the sheet pile. These tieback locations were specified to be spaced horizontally along the wall on eight foot centers. A small cavity was dug out from behind the wall at each hole to aid in the initial installation of the helical tieback, plus allow some room behind the wall for connections later in the construction sequence.

The helical anchor was installed at a thirty degree batter below the horizontal. Anchor installation continued until the proper torque and depth was achieved. The average free length above the top-most helix was twenty five feet. Installation torques ranged from 7,000 to 10,000 ft.-lbs. Torque was estimated by using a hydraulic pressure gauge which measured system pressure in the hydraulic circuit used to drive the torque head. Upon reaching the required torque and depth, a one inch diameter Dywidag bar was attached to the anchor shaft through the use of a Dywidag bar adapter.

Next, a plate was welded on with about a two inch slotted hole to cover the fourteen inch torched hole in the sheet pile wall. Side plates were welded on wall at each tieback location, one on each side. A waler consisting of two channels welded back-to-back with a gap in between was installed next. The waler usually was installed in sections of twenty feet. All sections were welded together to form one continuous waler for the entire length of the wall. Figure 2 details a wale assembly and the connections thereto.

After installation of the two-channel waler, the helical tiebacks were performance or proof tested as described later. Load was applied with a Dywidag bar stressing jack and pressure gauge. Tests were usually conducted after about twenty feet of the two-channel waler was installed. The load was locked-off at waler at 0.75 times the design load with a washer-plate and nut. Washer-plate was a four by six by one inch thick galvanized steel plate while the nut was a galvanized one inch Dywidag nut.

Figure 1. Overhead view of the job site and soil boring data.



DEPTH (FEET)	STD. PENETRATION PER ASTM D-1586 BLOW COUNTS (N VALUES)	SOIL DESCRIPTION
1	5	RED CLAY
5	13	RED MICACEOUS SILTY CLAY
10	7	YELLOWISH BROWN - VERY MICACEOUS SILT
15	6	BROWN - VERY MICACEOUS CLAYEY SILT
20	40	BROWN FINE-SANDY MICACEOUS SILT
24	50	NO RECOVERY
26	REFUSAL	BEDROCK

Figure 2. Wale Assembly

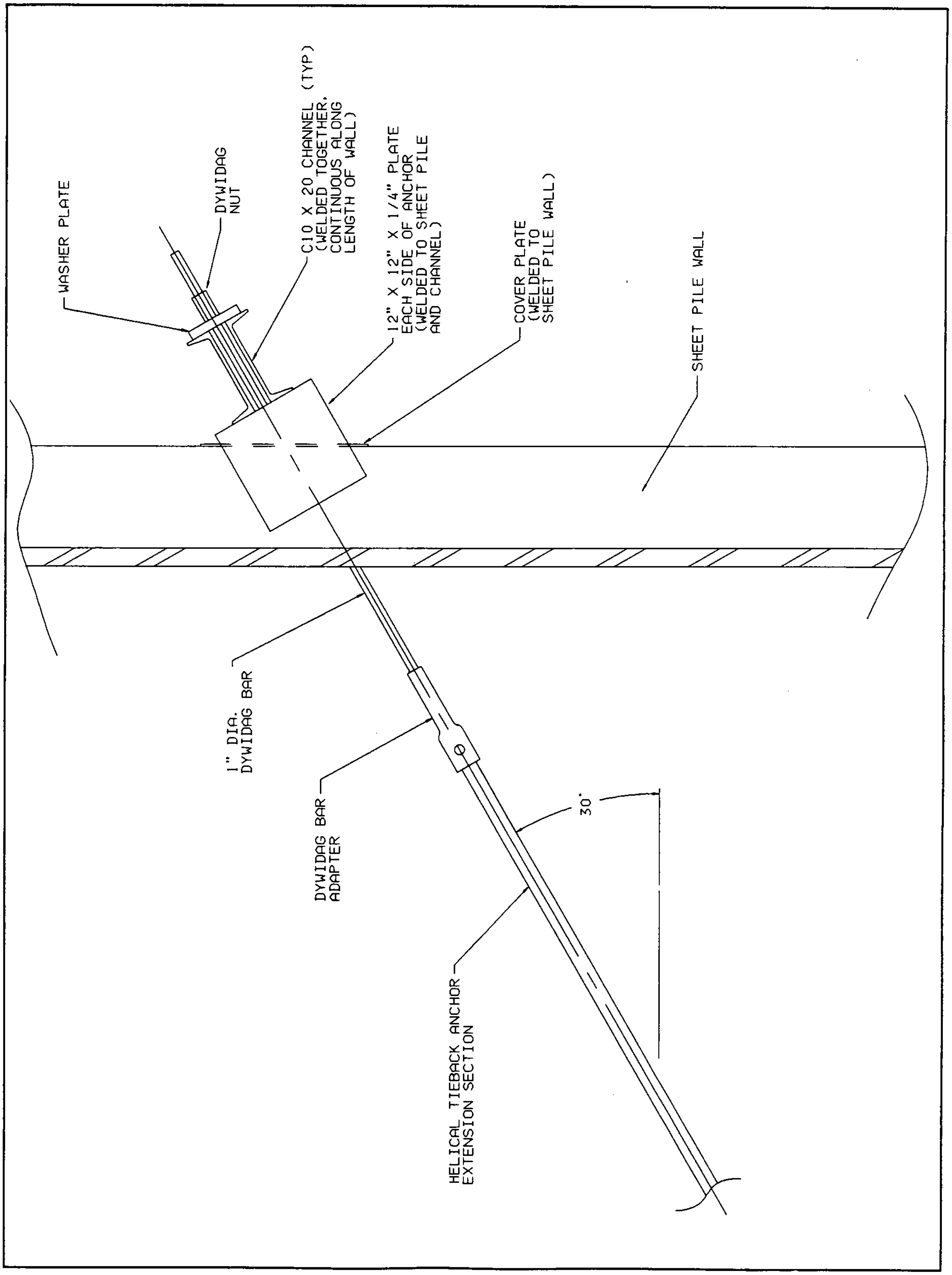
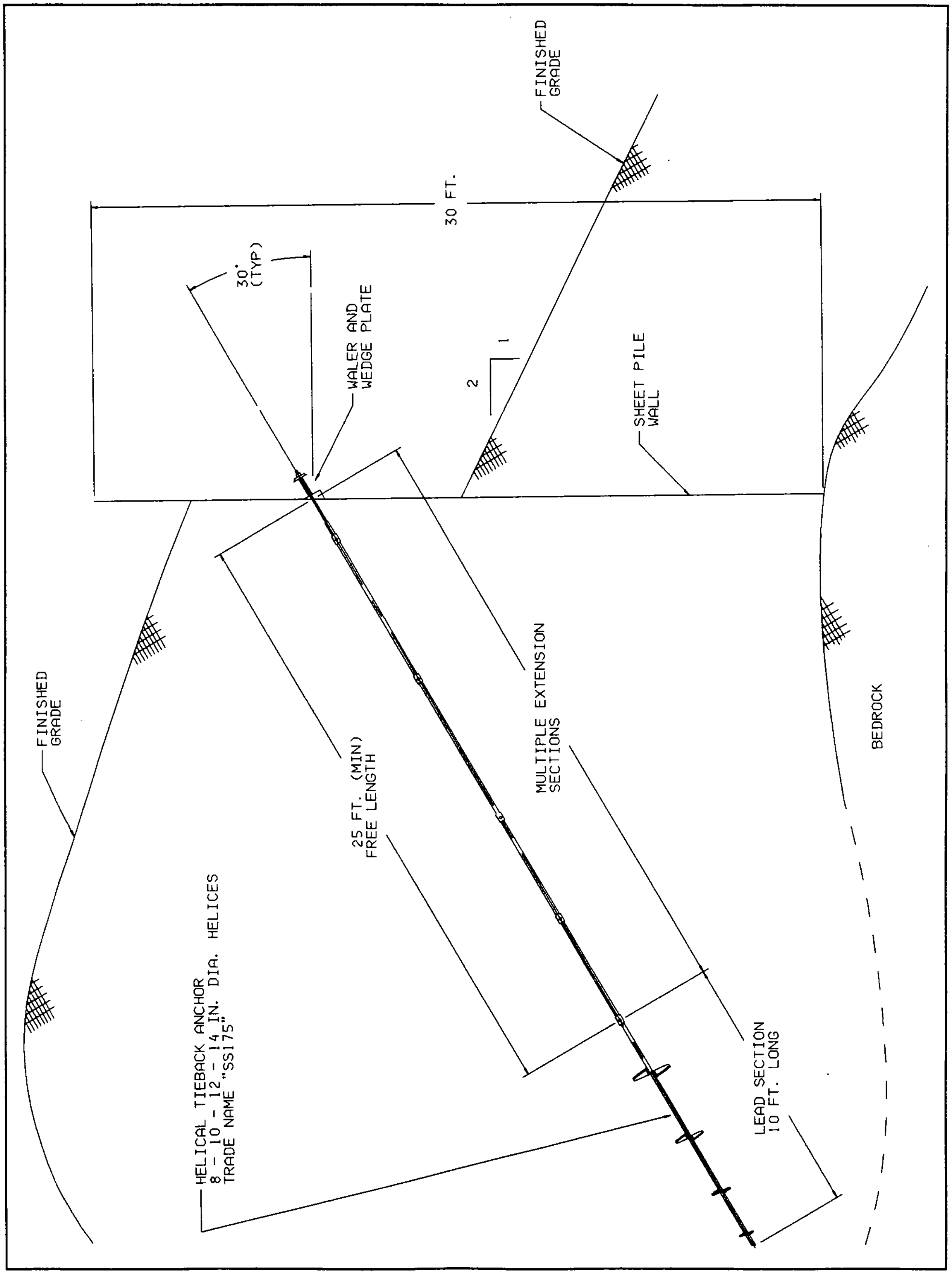


Figure 3. Typical wall cross-section



This sequence was continued for the entire 300 foot wall. After the wall was completed, grading and drainage work were done before the wall was to be sandblast and painted. This project required thirty four total tiebacks. A crew of three plus one equipment operator installed an average of five tiebacks a day. Figure 3 details a typical wall cross section showing the wall, tieback, and finished grading drawn to scale.

CONSTRUCTION SCHEME

The helical anchors were installed by using a conventional rubber-tired John Deere backhoe owned by the general contractor. When a section of wall was ready for tiebacks, a 10,000 ft.-lb. rated hydraulically driven torque head was mounted to the arm of the backhoe. The mount was a fabricated steel assembly with tabs that accepted the bale of the torque head. This mount was welded to the end of the backhoe arm. Once mounted, the torque head simply rested on the back side of the bucket, which was pivoted so that the bucket was extended. After attaching the torque head to the mount, hydraulic power was connected to the head by an auxiliary line from the backhoe's own hydraulic system. This arrangement was a very convenient method to install tiebacks. To adjust the installation angle of the tieback, the backhoe operator simply adjusted the arm position and/or pivoted the bucket.

Accessibility was not a problem because the backhoe could reach high enough along the wall to install tiebacks. Helical anchor lead and extension sections are available in various lengths - so the backhoe never had to be far away from the wall.

After a series of tiebacks were installed, the torque head was disconnected and removed from the backhoe. The backhoe could then be operated without any restrictions of its normal use.

TIEBACK TESTING PROCEDURES AND RESULTS

Each tieback was tested. The testing equipment consisted of a dial gauge accurate to 0.001 of an inch to measure axial deflection of the tieback tendon. The pull test unit consisted of a hollow ram jack and pump to apply the load, plus a calibrated pressure gauge. The ram assembly rested on a jack stand that was placed on the waler at each tieback location. The waler distributed the jack reaction load along the wall.

The test sequence required the first three tiebacks for the sheet pile wall and five percent of the remaining tiebacks to be "performance tested". The remaining tiebacks were "proof tested". Testing was done in accordance with Post Tensioning Institute recommendation (PTI - 1986). The performance test was a load cycle test where load was incrementally applied and then removed. This was done to a maximum test load of 32 tons, or 1.33 times the design load. The proof test was an incrementally increasing load test where the load was applied in steps up to a maximum test load of 32 tons. In both the performance test and proof test, the maximum load was held for 1, 2, 3, 4, 5, 6, and 10 minutes. The total deflection with respect to a stationary point in front of the wall was measured and recorded with the dial indicator at each time and load increment. Additional time at the maximum test load was required if total movement of the tieback tendon was considered excessive. Upon completion of each tieback test, the load was "locked-off" at the waler. This lock-off load was equal to 0.75 times the design load, or about 18 tons.

Tieback tests were considered acceptable based on several factors - one of them being that the creep rate was not to exceed 0.08 inch per log cycle of time during the load hold of the performance or proof tests. After the tiebacks were accepted, the portion of the Dywidag bar protruding beyond the waler was cut to an appropriate length.

Results from some of the proof tests are shown in Figures 4 and 5. Figure 4 is a load-deflection plot that shows the total range of deflections measured up to the maximum test load of 32 tons. Several tests were plotted, so an envelope curve was created. Figure 5 shows some typical time-deflection curves for several tieback tests. The constant load was the maximum test load of 32 tons.

Since the helical tiebacks used for this project are permanent, an analysis of the time-rate of settlement of the helical bearing plates was done. Soil consolidation in front of the helices was analyzed using the Taylor method [Taylor, 1948]. This is an empirical method in which deflection data is plotted against the square root of time at various intervals. This

Figure 4. Load-deflection plot for several tiebacks

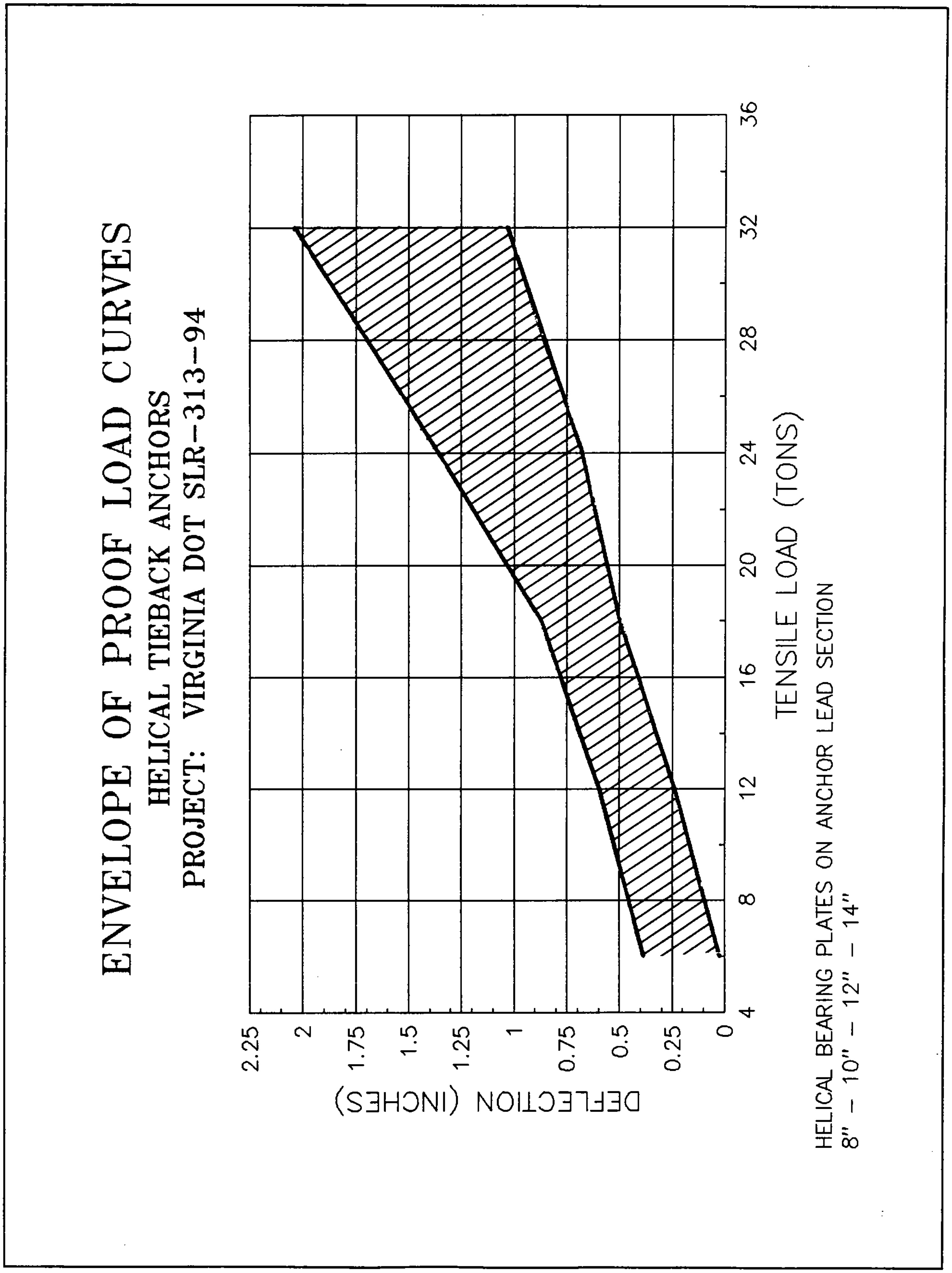


Figure 5. Time deflection plot - several tieback tests

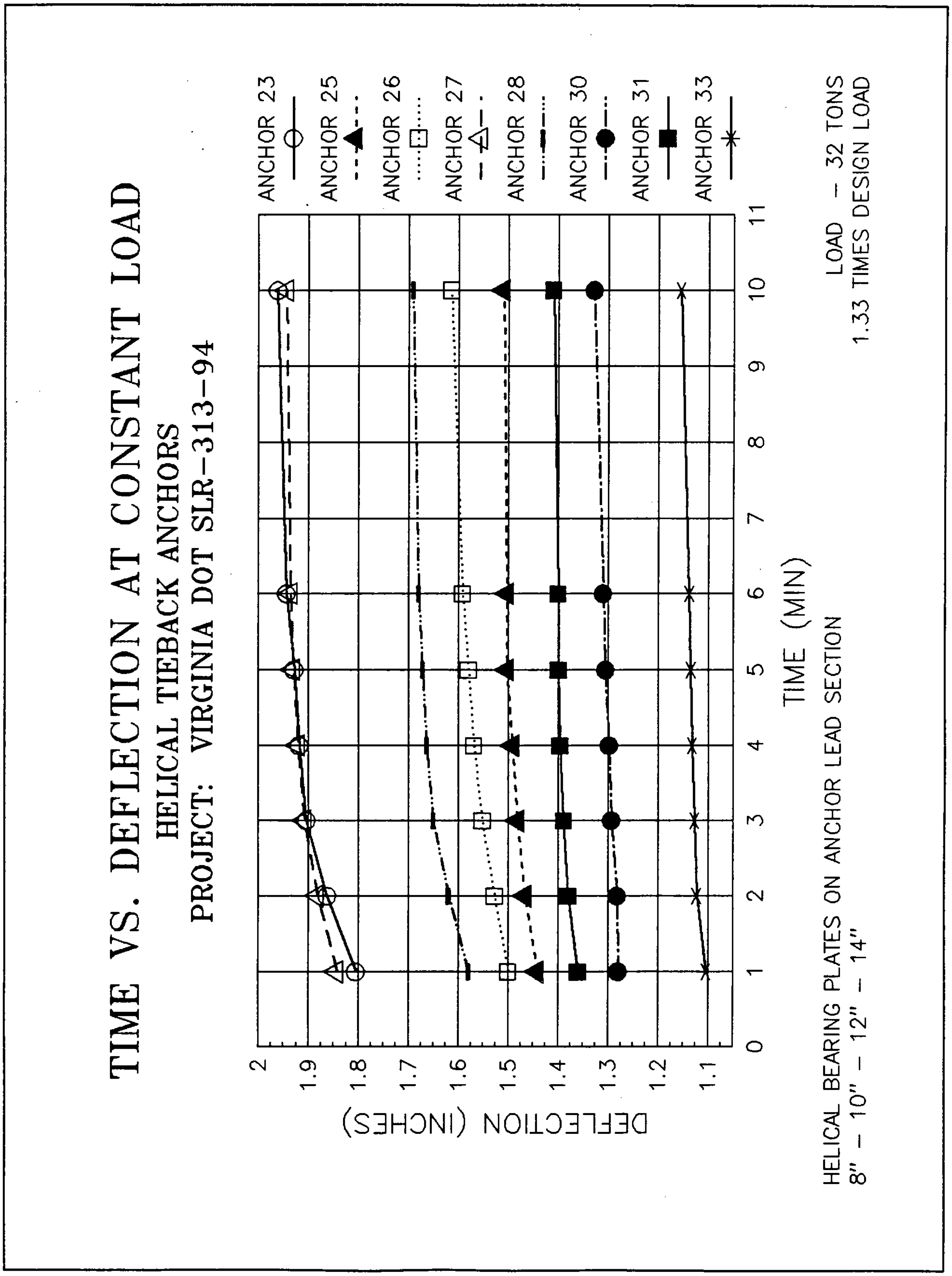
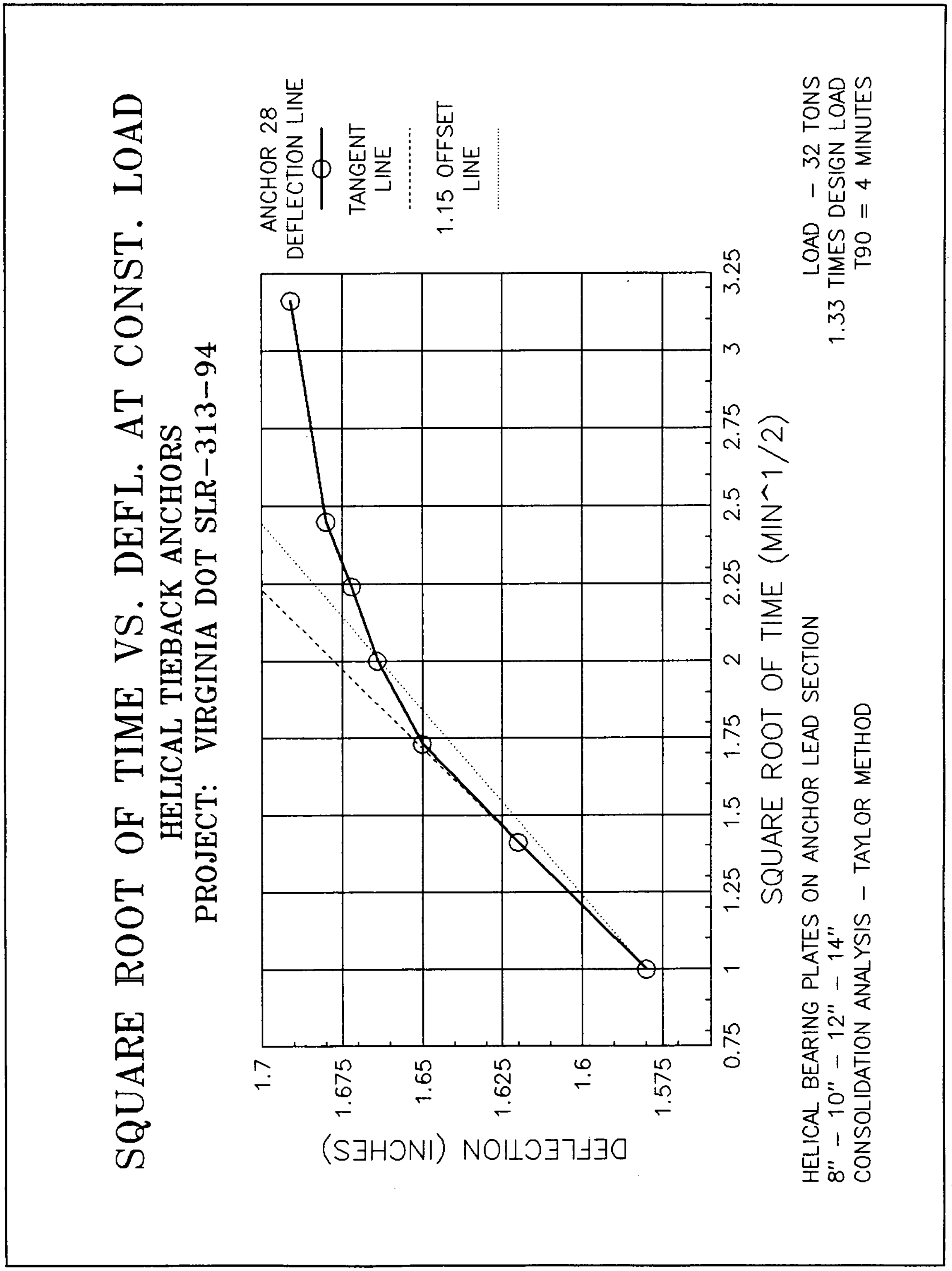


Figure 6. Taylor's square root of time fitting method



can be done at several increments of load. Figure 6 demonstrates the use of the Taylor method. The tangent line is a straight line corresponding to the slope of the early part of the test curve. The 1.15 offset line is a line in which for any point of deflection, the abscissa - or time in this case, is 1.15 times the tangent line. The intersection of the 1.15 offset line and the test curve is called the $\sqrt{t_{90}}$. The t_{90} is assumed to be the point at which ninety percent of the primary consolidation has occurred. Figure 6 shows a t_{90} value of four minutes for anchor number 28. Of the tests analyzed for this paper, t_{90}'s averaged about 4.5 minutes with the shortest being 2.6 minutes and the longest being 6.7 minutes. These low t_{90} values are probably due to the helical plates being embedded in the sandy silt.

The results confirm that helical tiebacks performed as expected and that future movement of the sheet pile wall should be minimized.

CORROSION

As with any permanent earth retainment wall construction, corrosion of the wall components is a consideration. Due to the non-corrosive nature of the soil on this project, galvanized helical anchor components were considered adequate for the tiebacks. As mentioned before, the exposed wall and waler are to be sandblast and painted when construction is complete.

SUMMARY

A sheet pile wall using helical anchors as tiebacks was shown to be a feasible, economic method of construction for permanent earth retainment walls. This project demonstrates the cost-saving advantages that helical anchors can provide. Minimal special equipment requirements, construction sequence flexibility, and access advantages are some of the reasons for this. Proper anchor selection and full scale testing have confirmed the feasibility of helical anchors as tiebacks for permanent retaining walls.

ACKNOWLEDGEMENT

This paper would not have been possible except for the contributions made by K. Y. Thrift, Jr., Vice President of J. A. Walder, Inc. Kavie's insight into the construction sequence and project history were invaluable. His help is very much appreciated.

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PERMANENT HIGHWAY AND LANDSLIDE WALLS

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ABSTRACT

The use of permanent tiebacks, soil nailing and rock bolting for highway walls and slope stabilization has increased significantly as Owners and the engineering community have accepted these systems as viable solutions to difficult engineering problems. The walls may be either predesigned by the Owner or design-build by the specialty contractor. Design-build has proved to be the most economical solution.

As with most engineered structures, there is more than one approach that will satisfy the technical requirements of a particular project. The design and construction sequence that provides the Owner the best value in terms of function, longevity, aesthetics, maintenance and cost is the one that should be selected. Many of the permanent tiedback, soil nailed, and rock bolted wall systems that maximize these values are proprietary and protected by patents—they are available to the Owner only through the design-build process.

This paper will review the design and construction of permanent highway walls and landslide walls with particular emphasis on applications in mountainous terrain that is indigenous to West Virginia. Three case histories will be presented to illustrate the use of permanent tiebacks, soil nailing and rock bolting for highway walls and slope stabilization. Each case history presents a unique and economical solution to a difficult engineering problem.

INTRODUCTION

The use of permanent tiebacks, soil nailing and rock bolting for highway walls and slope stabilization has increased significantly as Owners and the engineering community have accepted these systems as viable solutions to difficult engineering problems. The walls may be either predesigned by the Owner or design-build by the speciality contractor. Design-build has proved to be the most economical solution.

Permanent tiedback walls have been used in North America since the 1970's. The basic elements of a permanent tiedback wall are drilled-in or driven piles; temporary timber lagging; permanent corrosion-protected tiebacks; drainage material; and a permanent structural facing. Typically, the permanent facing is either cast-in-place, reinforced concrete or pre-cast panels attached to the piles.

Soil nailing/rock bolting has been used together with shotcrete to stabilized vertical cuts in a variety of applications. For remote, difficult to access sites, soil nailing/rock bolting has proved to be an economical solution. The basic elements of a soil nailed/rock bolted permanent wall are the soil nails/rock bolts; temporary shotcrete facing; drainage material; and a permanent cast-in-place, reinforced concrete, shotcrete, or precast facing.

This paper will review the design and construction of permanent highway walls and landslide walls with particular emphasis on applications in mountainous terrain that is indigenous to West Virginia. Three case histories will be presented to illustrate the use of permanent tiebacks, soil nailing and rock bolting for highway walls and slope stabilization.

Some of the information contained in this paper is condensed from previous papers (Anderson, 1984; Leichner, 1987 & 1988; Ludwig, 1988; Franceski and Ludwig, 1995; Fernworn, 1989).

PERMANENT TIEDBACK HIGHWAY WALLS

In comparison to conventional retaining walls, permanently tiedback walls completely eliminate foundation piles, large footings, and backfill (see Figure 1). In addition, the quantity of excavation and concrete are reduced when a tiedback wall is used. The savings, resulting from elimination or reduction of these items, far exceeds the cost of installing the permanent tiebacks, in many situations. Generally, in cut situations where temporary sheeting is required for conventional wall construction, permanent tiedback walls will be the most economical solution. Figure 1 shows that tiedback walls do not require wide construction easements.

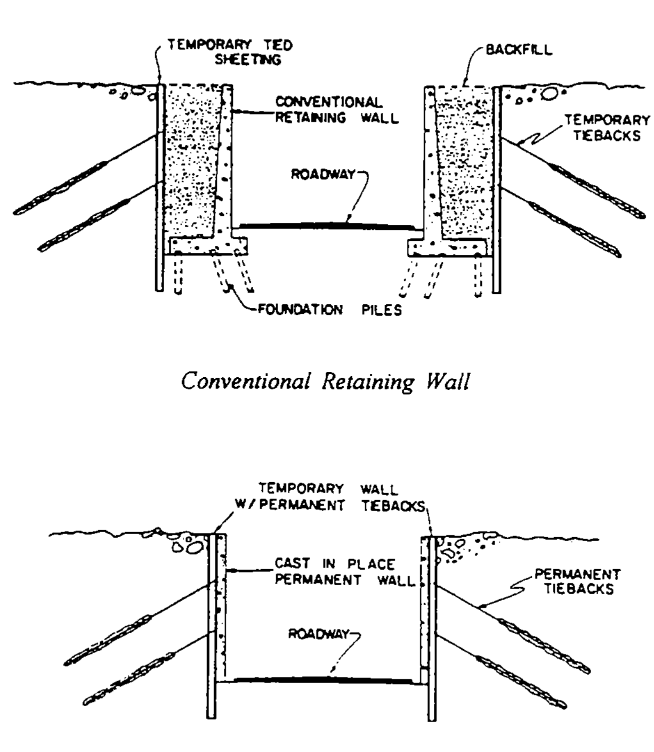


Figure 1. Wall Repair

DESIGN OF PERMANENT TIEDBACK HIGHWAY WALLS

Schnabel Foundation Company uses the earth pressure envelope (see Figure 2) for the design of most tiedback walls. It is very similar to the 1948 recommendations of Terzaghi and Peck. One difference is that this diagram is used for certain sand, clay or mixed soils without distinguishing between them. This is a direct result of supporting instrumentation data and the quality of the workmanship. For reasonable soil values, there is close agreement regarding the maximum unit pressure and total pressure against the wall between Schnabel's diagram, and Terzaghi and Peck's diagram (see Figure 2 for comparison). Schnabel's diagram is an empirical earth pressure distribution, based on measuring actual jobs, and it is used to design similar projects in similar soils. This diagram is not used blindly in softer soil or soils which may exert greater pressures. In addition, if unusual surcharges or water exert pressure on the wall, these must be added to the earth pressure.

Once the appropriate earth pressure diagram has been selected, the designer of the tiedback wall must then consider wall stability and mass stability. In order to be stable, the wall must be strong enough to confine the excavation when acted upon by the appropriate tieback forces. The unique feature of tiedback walls is that the tiebacks act to resist all or most of the earth pressure. Even though most walls penetrate below subgrade, it is assumed that no pressure acts on the wall below subgrade when using the envelopes shown in Figure 2.

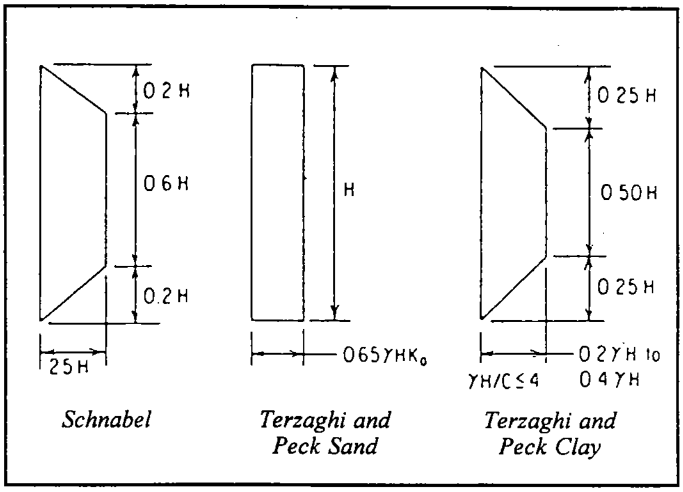


Figure 2.
Comparison of Several "Apparent Earth-Pressure" Envelopes

Tiebacks should not develop any of their capacity in the soil between the critical failure surface and the wall. Clearly, any capacity the anchor developed in this soil would be reduced if the wall moved. This is prevented by specifying the length of tieback which must be unbonded.

Tiebacks usually apply a downward force on the wall. Walls have failed as a result of not considering this condition; hence, this force should not be ignored in the design of the wall. Wall friction acts to decrease the downward component of the tieback force. Normally, in competent soils, axial loading of the wall is not considered until the angle of the tieback from the horizontal exceeds ϕ (angle of internal friction). When steeper tiebacks are necessary, the wall should be designed to develop resisting capacity below subgrade and carry this downward component to a depth where suitable bearing capacity can be achieved.

Tiebacks tie the wall to a wedge of soil which must be internally and externally stable. If the wall is properly designed and the tiebacks have the capacity for which they are designed, the wall pressures and tiebacks create internal forces in the wedge. The wedge must also be checked for external forces to verify its stability. The shape of this mass is fixed by the location of the tieback anchors, so a stable mass can be assured by properly locating the anchors.

WALL COMPONENTS

A typical permanent tiedback wall system is shown in Figure 3 and is comprised of soldier beams, lagging, tiebacks and facing. The soldier beams may consist of H-piles, wide flange shapes, reinforced caissons or sheet piling. Depending on the soil conditions the piles may be driven or set in drilled holes and filled with lean concrete. Lagging is installed between soldier beams as the excavation proceeds from the top down.

Space is left between lagging boards so that drainage is promoted. Tiebacks are installed after the excavation and lagging is complete down to a predetermined elevation. Tiebacks may be pressure injected, hollow-stem auger, regroutable, rock tiebacks or some other type based on the geotechnical site conditions. The process of excavating, lagging and installing tiebacks is continued until the desired subgrade is reached. In permanent walls, adequate facing and drainage provisions are required to ensure satisfactory long-term performance.

Permanent facing systems for anchored structures should be compatible with the construction sequence of the structure and provide the required structural capacity for the life of the structure. In addition, the permanent facing should be aesthetically pleasing. Adequate drainage must be provided with all systems in order to prevent hydrostatic pressure from acting on the facing.

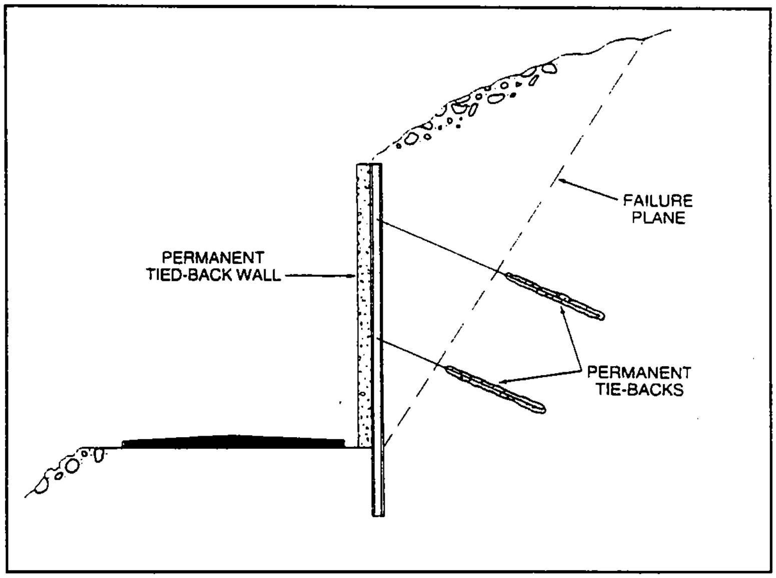


Figure 3. Permanent Tiedback Wall System

Many permanent tiedback walls have used precast concrete planks or panels between soldier beam flanges to act as both lagging and permanent facing. The problem with this detail is that since tiedback walls are built from the top down, the planks or panels are installed prior to stressing the anchors. When the anchors are stressed, the soldier beams are pushed back against the soil. This movement and associated earth pressure cause the planks or panels to bow outward and crack. Structurally and aesthetically, performance such as this is clearly undesirable. Another aesthetic shortcoming of this facing is that the pile flanges and tieback anchorheads are exposed.

Improved performance and appearance can be achieved by using temporary timber lagging and providing precast facing panels or cast-in-place reinforced concrete to conceal the anchorheads, soldier beams and lagging boards. This system allows the facing to be installed from the bottom to the top after the anchors have been stressed and movement associated with stres-

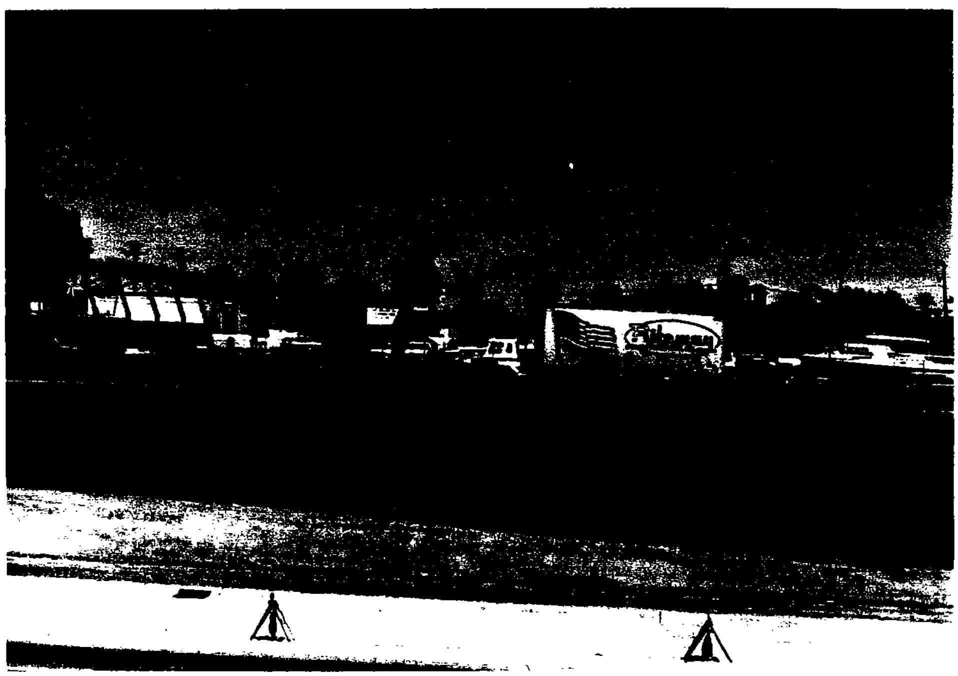


Photo 1.
Tiedback Wall Built With a Cast-in-Place, Reinforced Concrete Facing

sing has occurred. Photo 1 shows a tiedback wall built with a cast-in-place, reinforced concrete facing, while Photo 2 shows a tiedback wall built with precast panels without exposed wales and anchorheads.

Photo 2.
Tiedback Wall Built With Precast Panels Without Exposed Wales and Anchorheads

CASE STUDY — BRIGHAM CITY AND MANTUA, UT

For many years, Utah State Route 91, a two-lane road between Brigham City and Mantua, has been the scene of many serious accidents and delays. To eliminate these problems, the Utah Department of Transportation (UDOT) wanted to widen the road and improve its alignment. Since Box Elder Creek, a Class 3 (high priority) fishery ran adjacent to Route 91, extensive precautions would be necessary to minimize disturbance to the stream. In some areas large cuts and substantial fills were made to obtain the desired geometry. In other areas the limited right-of-way and steep hillsides required the construction of mechanically stabilized earth embankments and retaining walls in cuts.

The most difficult section required excavation into a steep talus slope on the south side of Box Elder Canyon. To prevent a massive over excavation into the hillside, the UDOT planned to construct a tiedback wall. The difficulty of constructing a 600-foot-long wall with a maximum height of 40 feet, in a steep canyon with cobbles and boulders made the construction of a traditional soldier beam tiedback wall impractical. To find an economical and constructable solution for this road widening project, the Utah Department of Transportation decided to let this project as a design/build contract. This process involved soliciting preliminary proposals from several design/build retaining wall contractors. Then the specialty contractors with acceptable proposals were asked to submit complete design drawings and calculations for review by Centennial Engineering, DOT and FHWA. Once accepted, these designs were incorporated into the bid documents for the overall project. This allowed the General Contractors to choose the speciality contractor's design/build proposal that would result the in most constructable and economical contract.

Le Grand Johnson was the low bidder with the wall designed by Schnabel Foundation Company. Schnabel Foundation Company's design incorporated shotcrete and soil nails to temporarily stabilize the slope and corrosion-protected tieback anchors, cast-in-place wales, and precast facing panels to provide the long term support of the cut. This system eliminated the need to install soldier beams. The tiedback wall was completed on schedule, with no claims, and the project received Utah's Department of Transportation 1994 Award for its contribution to transportation excellence. This engineering solution to a complex problem clearly illustrated the benefits of a design-built performance specification over a prescription specification.

In general, the existing slope at the wall location ranged from 35 to 50 degrees. The face of the hill consisted of a boulder-cobble colluvium with a silty, clayey, sand matrix. Several areas across the slope were covered with talus slides composed of quartzite cobbles and boulders. The subsurface profile, as determined by a representative boring at this location, consisted of quartzite and quartizitic sandstone cobbles and boulders, supported by a clayey sand matrix above quartzite bedrock encountered 45 to 70 feet below the existing grade at the wall location. Boulders up to 3 feet in diameter were encountered in the boring. The amount of matrix appeared to vary. A typical section through the tiedback wall is shown in Figure 4.

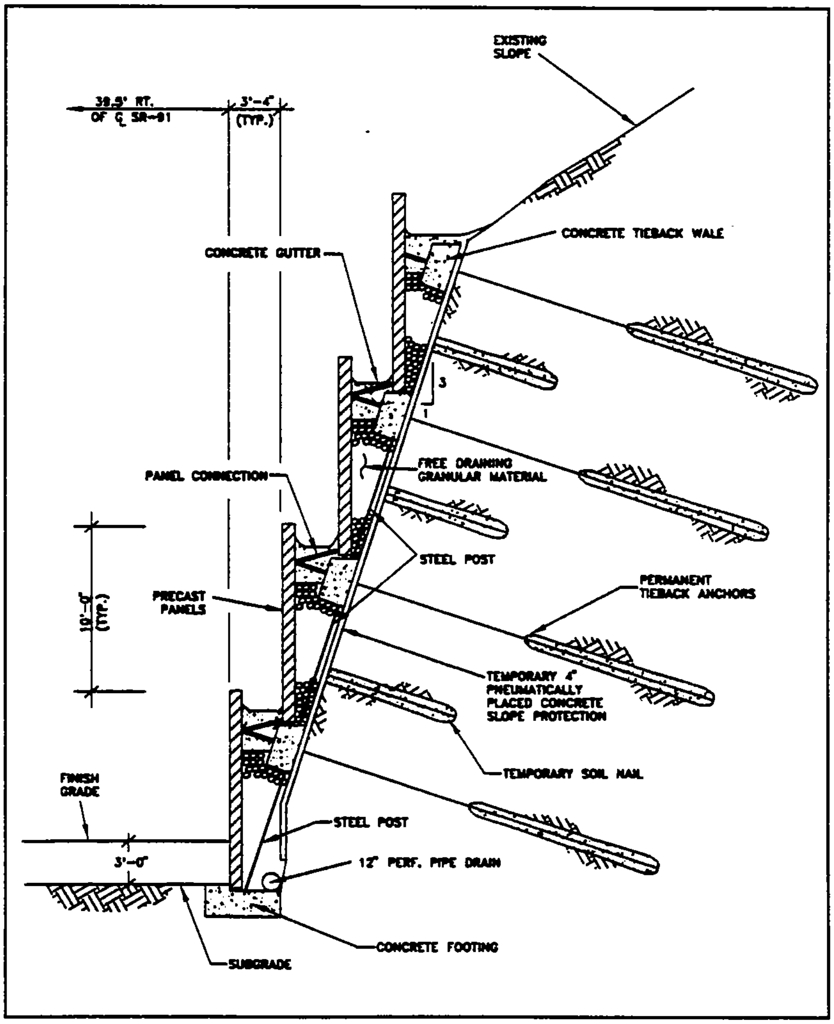


Figure 4. Section Through Wall

The primary elements of the wall are: permanent tiebacks, horizontal, cast-in-place, reinforced, concrete wales; precast panels; cast-in-place, reinforced, concrete footing; connections from the precast panels to the wales; and steel posts spanning from wale to wale. Shotcrete and soil nails were installed as required to provide temporary shoring during the excavation of the cut slope. A patent is pending on this system.

The tiedback wall was designed using a rectangular earth pressure diagram with a pressure of 30H psf (H=height of cut). The maximum pressure of 30H was determined by calculating the total force resulting from the recommended equivalent fluid pressure, provided by the geotechnical engineer (i.e., triangular pressure distribution), and converting the force to an equivalent rectangular pressure. The tieback design force was calculated based on the 30H design pressure, the wale spacing of a 10-foot maximum and tieback spacing of a 10-foot maximum. The tiebacks were designed to resist the full earth pressure. The reinforced concrete wale was designed as a continuous beam having a maximum span of 10 feet. The maximum wale size was 3 feet wide by 1.5 feet deep. The wale size and amount of flexural and shear reinforcement varied as a function of the height of cut. The precast panels were designed for the greater pressure of 80% of 30H psf, or 1000 psf. The 80% factor is an effective arching factor which takes into account some degree of soil arching which will occur between the wales. The pressure was assumed to act over a height equal to the clear span from wale to wale. The footing was designed to support vertical load from the panel and backfill, and the load in the tube post, as well as lateral loads from the full earth pressure acting on the bottom half of the lowest panel and the lateral thrust from the tube post embedded in the footing.

The first step in constructing the tiedback wall was to build a bench from which the top row of tiebacks could be installed. The next step was to make a cut from the proposed excavation line on a 1:3 batter. Immediately after making the excavation, prefabricated geocomposite vertical drains were placed at 5 feet on center. Next, welded wire fabric and a 4-inch nominal shotcrete facing was applied. After the shotcrete was placed and rough finished at the wale location, the tiebacks were installed. All tiebacks were tested to a minimum of 133% of the design load. After the tiebacks were installed, but prior to testing, the reinforced concrete wales with the required embeds were cast around these tiebacks tendons. At each tieback location an additional sleeve was cast in the wale to allow drilling a supplemental or replacement tieback if the original one failed to hold the test load. When the concrete in the wales reached the 3000 psi design strength, the tiebacks were tested and locked off.

The vertical distance between wales is 10 feet and after the anchors were locked off, a 5-foot cut was made where the ground would stand (less if not) and shotcrete placed. Because the distance between the bottom of the previous wale and the bench for the next tieback/wale level was 12 feet, short soil nails were installed midway between wale levels to stabilize the slope as the excavation proceeded. The sequence of installation of shotcrete, soil nails, tieback anchors, and cast-in-place wales was repeated until wall subgrade was reached.

At wall subgrade a 48-inch by 18-inch continuous strip footing was cast to carry the vertical load of the wall system. The analysis of all the vertical forces imposed by the vertical component of tieback force, the weight of wale, precast and backfill (minus the resisting frictional force on the back of the wales), produced a net downward vertical force which was carried by columns or posts. These posts were designed to carry the vertical load down from wale to wale and eventually into the footing. The posts were structural steel tubes welded to plates embedded in the wale. In order to protect the posts and plates from corrosion, they were encased in shotcrete after being welded in place.

After the footing was cast and reached design strength, erection of the precast facing panels was begun. Up to this point the temporary shotcrete and soil nails carried the earth pressure between successive wale levels. The final step in completing the load transfer from the earth to the wales was the erection of precast panels to span vertically between wales. Gravel backfill was placed between the precast facing panels and temporary shotcrete. Photo 3 shows the 90% completed wall.

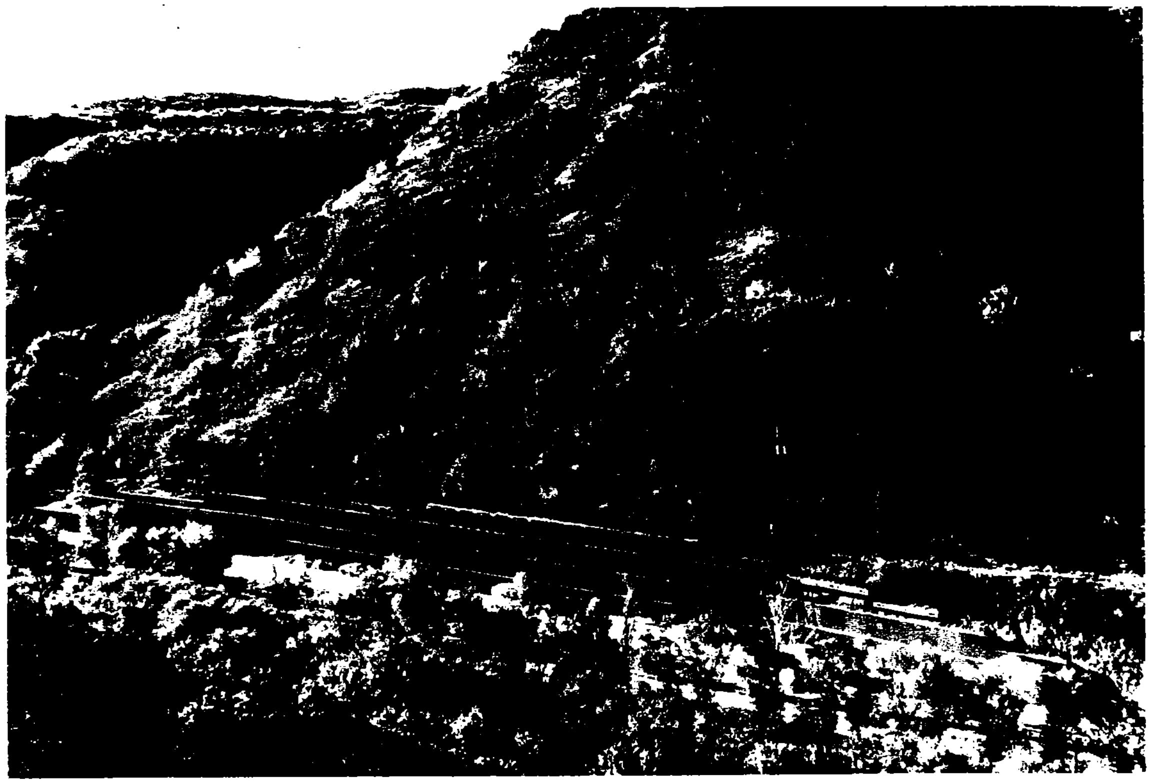


Photo 3. Semi-completed Wall

The method of contracting gave the Owner the benefit of several Speciality Contractors' expertise in the design and construction of tiedback wall systems. As with most engineered structures, there is more than one approach that will satisfy the technical requirements of a particular project. Logically, the design and construction sequence that provides the Owner the best value in terms of function, longevity, aesthetics, maintenance and cost is the one that should be selected. With tiedback walls, many of the systems that maximize these values are proprietary and protected by patents and are available to the Owner only through the design-build process. Additionally, the Owner can control product quality and design through the review process, and ensure a competitive price through the process of competitive bidding.

SOIL NAILING/ROCK BOLTING

Soil nailing has been used as a lateral earth support system since the early 1970's. The technique involves reinforcing the ground by placing tension elements, as the excavation proceeds downwards, to increase the overall shear strength of the in situ soil. The tension elements are usually steel bars that are driven or drilled, and grouted in place. A relatively large number of soil nails are installed in a pattern that reinforces the ground into a stable block that behaves similar to a gravity retaining wall. Figure 5 shows a schematic view of a typical soil nailed wall.

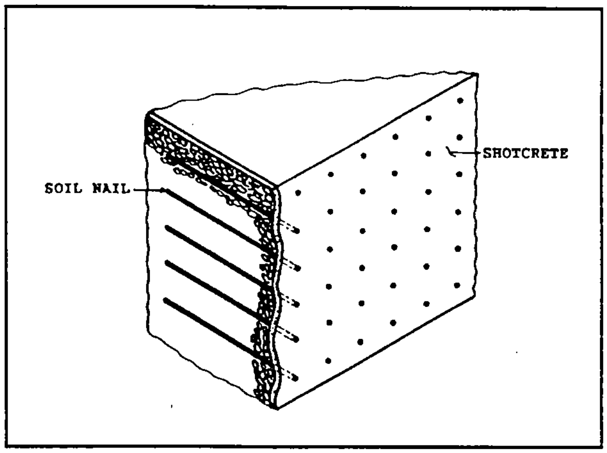


Figure 5
Soil Nailing Reinforces the Soil into a Block

Soil nailing has been used in a wide variety of applications where a conventional or anchored retaining wall would have been less effective. The primary requirement is that the ground is able to stand unsupported, while the nails are installed and a facing is applied. Design and installation methods have been developed primarily by specialty contractors. While soil nailing is not patented, many of the construction techniques and details are patented or proprietary.

The design method currently used by Schnabel Foundation Company is SNAIL developed by CALTRANS. It is a limit equilibrium method of analysis which computes the driving forces (due to the weight of the soil and any external surcharges) and resisting forces (due to the shear strength of the soil and the resistance provided by the soil nails). Factors of safety are applied to the shear strength parameters of the soil.

The spacing and length of the nails is dependent on in-situ soil properties and wall geometry, which can vary from those assumed, requiring adjustments in the field. The timing of placing the facing material and drainage behind the facing can be critical. Consequently, a designer must recognize that the soil is the structural system. The reinforcements and drainage provisions are meant to improve the strength of the structural system, but the contractor's methods of placement and timing could have an unexpected and detrimental effect on the system's performance.

The best situation is when the contractor has responsibility and control of the design, along with successful prior experience working with the type of soil specific to the site. A performance specification is recommended when contracting for soil nailing work, giving the contractor control of both the design and construction with a quality level defined for the final product. Certain information that should be provided by the specifier in a performance specification are:

- Provide a detailed geotechnical site investigation. The normal parameters required are cohesion, friction angle, unit weight, moisture content, allowable bearing pressure and standard penetration resistance.

- Specify monitoring and testing requirements. An optical survey and proof testing of 5% of nails is most commonly done.

- For a permanent soil nailing wall, describe: the level of corrosion protection required, the wall facing and drainage that will be acceptable, and any standard specifications to be used in designing the wall.

The majority of soil nailed walls constructed as permanent retaining walls have been built with a separate concrete facing using precast or cast-in-place concrete.

CASE STUDY — EASTON, PA

Near the end of a 400,000-cubic-yard rock cut on a section of I-78, the general contractor encountered a 360-foot-long section of badly weathered sandstone, soil and boulders. Much of this material, which had not been identified in the soils investigation, would crumble like "dried coffee grounds" when touched. Obviously, this section would not stand as a rock cut.

The limited access to the site and the presence of large boulders would significantly increase the cost of retaining wall construction. Also, the construction of a conventional wall or a mechanically stabilized embankment would have required an excavation that would reach hundreds of feet up the hillside and been unacceptable in this environmentally sensitive area.

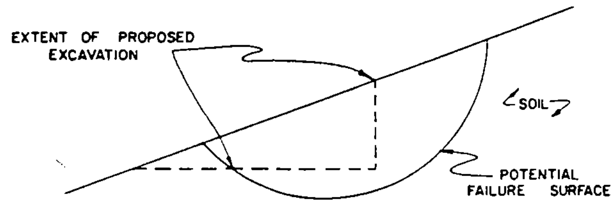
PENNDOT investigated the possibility of constructing a cantilevered retaining wall, but estimated that it would cost over a million dollars to construct. PENNDOT then contacted consultants and specialty contractors to evaluate the possibility of constructing a tiedback wall. The cost of constructing a tiedback wall in this material would be high—a more feasible and economical solution would be the construction of a soil nailed retaining wall.

Construction began with the excavation of a shallow cut. In many sections the excavator could only remove two to three buckets full at a time before the cut had to be stabilized. The cut was stabilized by placing reinforcing mesh, installing drains, and shotcreting. Once the shotcrete had set up, an air track was used to drill small diameter holes for the soil nails. These holes were then filled with grout and the nails were installed. The next day a plate was attached to each nail. To construct the second and successive lifts, an additional five feet of material was removed immediately underneath the soil mass that had just been stabilized by the soil nails and shotcrete. This procedure was repeated until the excavation was completed. The excavation and soil nailing process was made quite difficult because of large boulders, cavities and other features that made a clean excavation impossible. As soon as subgrade was reached, a leveling pad was poured on which the precast panels would be set. Precast panels were then attached to the plates and held in place by the soil nails. As the panels were set on top of each other and connected to the nails, backfill was placed to fill the irregular void behind the panels and to provide drainage. Photo 4 shows the completed wall.

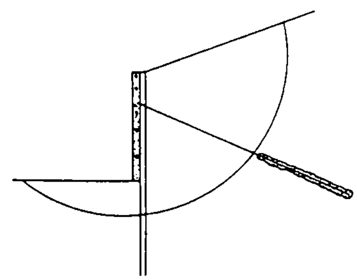
Photo 4. Completed Wall

LANDSLIDE STABILIZATION

Permanent tiebacks have been used effectively to stabilize or to prevent landslides. Permanent tiedback walls can be constructed to stabilize cut and fill slides associated with highway and railroad construction. Often these walls can be build without disrupting traffic on the existing roadway or railroad. Permanent tiedback walls also are used to prevent slides from occurring, as well as allow structures to be built into a potential slide area. As shown in Figure 6 a landslide may develop when an excavation is made into a hillside. Tiedback walls will enable the maximum development of these types of sites. In landslide applications, the permanent tiebacks extend below the failure surface and provide the force required for equilibrium. Limit equilibrium methods of analysis are used to determine tieback loads to provide equilibrium. Schnabel Foundation Company currently uses STABLV developed at Purdue University to analyze tieback applications for landslides.



Excavation Could Cause a Landslide and Site Normally Would not be Developed



Permanent Tiebacks Allowed this Site to be Developed

Figure 6.
Site Development Slope Stabilization

CASE STUDY — ALTOONA, PA

The new interstate construction of U.S. 220 connects Bedford and Tyrone Boroughs in south-central Pennsylvania. The primary reason for this construction was to tie Altoona into the National Interstate System. Part of the excavation for the project required a deep cut into the side of Brush Mountain for one of the exit ramps, where exposed, weak claystone layers in the cut slopes caused instability.

Stability analysis indicated a stable cutslope rate along the plane of the exposed weak layers would necessitate removal of much of the mountainside with massive excavation quantities and high costs. Other methods of stabilizing the hillside were considered, including a tiedback retaining wall at the base of the slope, creating a shot-in-place buttress in the bottom of the slope, or a system of rock tiebacks. The rock tiebacks were selected as the most cost effective at about half the anticipated cost of mass excavation to lay back the slope.

A total of 338 tiebacks were installed in a 6-inch hole, drilled at a 30-degree angle to a depth of up to 65 feet into deeper laying sandstone. They were secured at the top with precast concrete bearing blocks set into surface rock. The design load of each tieback was 120 kips. Five rows of tiebacks were installed. The entire rock-anchored reinforced cut slope stabilized the hillside, with the tiebacks increasing the normal force in the exposed rock layers. The end result is an interesting, even somewhat attractive hillside (see Photo 5). The total cost of the tieback system was about $700,000.

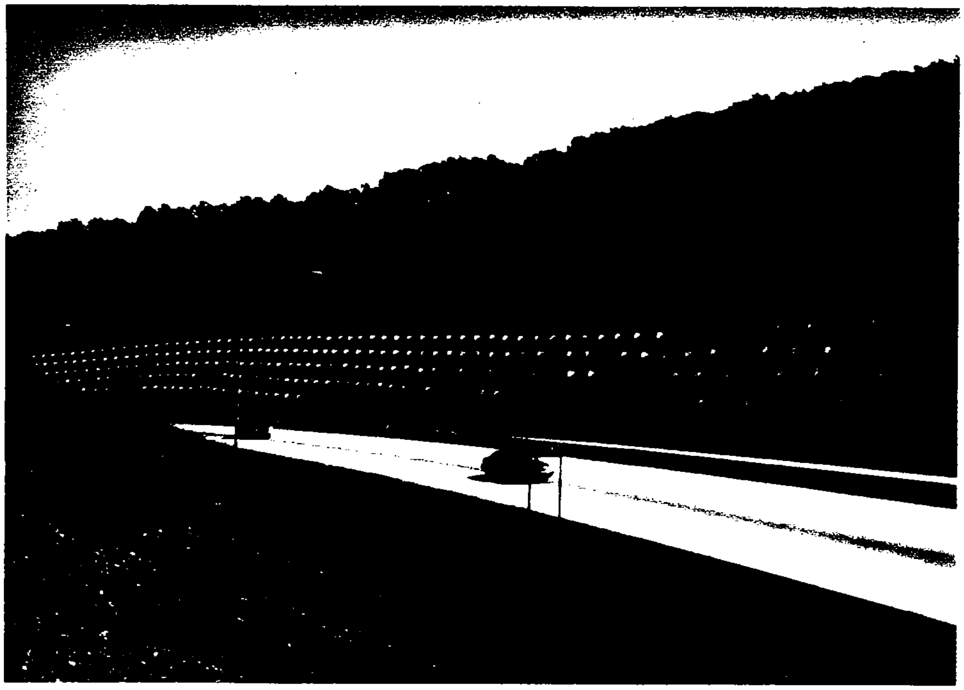


Photo 5. Slope Stabilization in Altoona, PA

CONCLUSION

The use of permanent tiebacks, soil nailing and rock bolting for highway walls and slope stabilization has increased significantly as Owners and the engineering community have accepted these systems as viable solutions to difficult engineering problems. As with most engineered structures, there is more than one approach that will satisfy the technical requirements of a particular project. The design and construction sequence that provides the Owner the best value in terms of function, longevity, aesthetics, maintenance and cost is the one that should be selected. With permanent tiedback, soil nailed, and rock bolted walls, many of the systems that maximize these values are proprietary and protected by patents—they are available to the Owner only through the design-build process. The Utah and Pennsylvania Departments of Transportation walls are both excellent examples of the benefits of allowing specialty contractors to design-build the two walls, thereby resulting in the most constructable and economical solutions to difficult engineering problems.

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Abandoned Deep Mine Subsidence Investigation And Remedial Design, Interstate 70, Guernsey County, Ohio

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ABSTRACT

A two thousand linear foot, undermined section of Interstate 70 in Guernsey County, Ohio has experienced significant recent settlement from pothole type subsidence events within the traveled lanes, shoulders and adjacent right-of-way areas. Several potholes measured approximately ten feet deep and ten feet in diameter. The subsidence occurred after the dewatering of the abandoned deep mine during auger mining operations ½ mile west of the site.

A two-phase emergency investigation was undertaken by the Ohio Department of Transportation (ODOT) and Gannett Fleming Corddry & Carpenter (GF). The purpose of the investigation was to assess the immediate danger of potholes occurring in the traveled lanes and paved shoulders, to identify the mechanism(s) causing the subsidence and to design a remediation program. Phase one investigations involved the review of existing subsurface information, the performance of multiple geophysical surveys, and the completion of test borings. The Phase one investigations did not reveal the presence of any subsidence voids.

Phase two investigations included drilling through the mine interval and videotaping mine conditions. Groundwater level measurements showed the mine was completely flooded. Based upon the collected data, two mechanisms of subsidence were identified, i.e., mine roof failure and piping of overburden soils into mine voids.

Two potential remedial schemes, the filling of the mine and the reinforcement of the highway with geotextiles, were evaluated. Filling of the mined interval and grouting of overburden bedrock fractures and voids, were selected. Construction plans, specifications and cost estimates were prepared for bidding and award.

PHYSICAL SETTING

Project Location The project involves approximately two thousand linear feet of Interstate 70 in Guernsey County, Ohio, approximately four miles east of Cambridge. Guernsey County is in east central Ohio, approximately 90 miles west of Pittsburgh, Pennsylvania and approximately 100 miles east of Columbus, Ohio.

Local Topography The affected portion of the highway is a flat, tangent section that is approximately nine thousand feet in length, lying within a broad, level valley that ends in steep sloping sides. The valley is drained by Mud Run. Two surface water reservoirs are located upgradient of the Interstate right of way.

GEOLOGY

Area Geology The area is located in the Appalachian Plateau physiographic province. As is typical to this province, this area is characterized by relatively flat or gently sloping sedimentary rocks and gently plunging folds. Formations in this area have been exposed to extensive erosional forces from water and gravity, resulting in well defined valleys and high order river systems. The topography in the plateau is typically narrow valleys with steep, unstable colluvial slopes.

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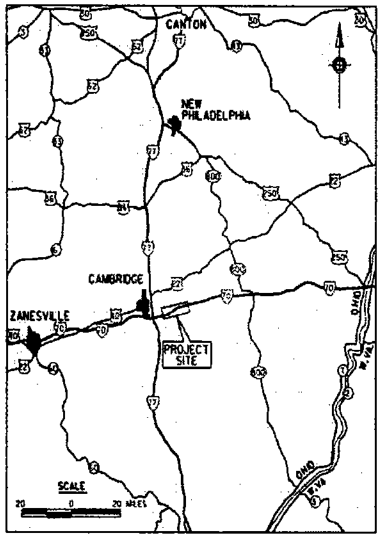
FIGURE 1: PROJECT LOCATION PLAN



Stratigraphically, the area bedrock is situated in the Conemaugh Group. The Conemaugh is of Pennsylvanian age and is generally divided into two formations: The Casselman and the Glenshaw. Prominent markerbeds in the area include: The Pittsburgh Coal, the lower boundary of the Monongahela Group; the Ames Limestone, the upper boundary of the Glenshaw Formation; and the Upper Freeport Coal, the upper boundary of the Allegheny Group. A geologic map is provided as Figure 2.

Much of the region has been directly and indirectly affected by glaciation. Direct deposit of till by glacial ice flow has occurred over most of Ohio. However, the area discussed herein saw only limited direct glaciatiaton. Instead, much of the area was affected by periglacial erosion and depositional processes. The result is largely stratified deposits caused by glacial outwash, lakes, flooding, frost action and solifluction.

<u>Site Geology</u> The project site is located approximately 30 miles south and east of the closest termination point of Wisconsinin and Illinoian glaciation. Soils at the site are stratified, with evidence that they were affected by the periglacial processes previously discussed. These soils tended to be clays and silts, with minor lenses of sands and gravels mixed in, especially in the deeper sediments. Many of the sediments were noticeable varved and contained varying amounts of interbedded and intermixed sands and gravels. Most of the site has a thin sand and gravel layer directly on top of bedrock. A generalized subsurface profile is presented as Figure 3.

Stratigraphically, the site is located in the lower Glenshaw Formation of the Conemaugh Group, just above the Upper Freeport Coal. Refer to Figure 4. Bedrock correlates as the Mahoning Sandstone and consisted largely of shale, sandy shale and sandstone. The regional strike is N30E and dip is less than one degree to the southeast.

<u>Coal Geology</u> A number of prominent coal seams of economic value exist in the area. Most of these coals are referred to locally by a numbering system as well as their common formation name. The most commonly mined seams include: Meigs Creek (No. 9 coal), Pomeroy (No. 8a coal), Pittsburgh (No. 8 coal), Mahoning (No. 7a coal), Upper Freeport (No. 7 coal), Lower Freeport (No. 6a coal), Middle Kittanning (No. 6 coal), Lower Kittanning (No. 5 coal), Clarion (No. 4a coal) and Brookville (No. 4 coal). Locally, the Upper Freeport Coal is mined using surface and deep mining techniques. The Upper Freeport is a good quality, low sulfur coal, but inconsistent in aerial extent due to numerous sand channels that were cut during deposition.

MINING HISTORY

<u>Deep Mining History</u> Mining occurred below the site in the Upper Freeport coal seam in the Murray Hill No. 2 Mine. Mining was discontinued in the mine in 1935 and in the adjacent King's Mine in 1927. The Murray Hill No. 2 mine was developed using room and pillar methods and appears to have been designed for secondary (total extraction). Main and butt entries were oriented at N11E and N79W, respectively, and were generally spaced like

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FIGURE 2: GEOLOGIC MAP AND CROSS SECTION

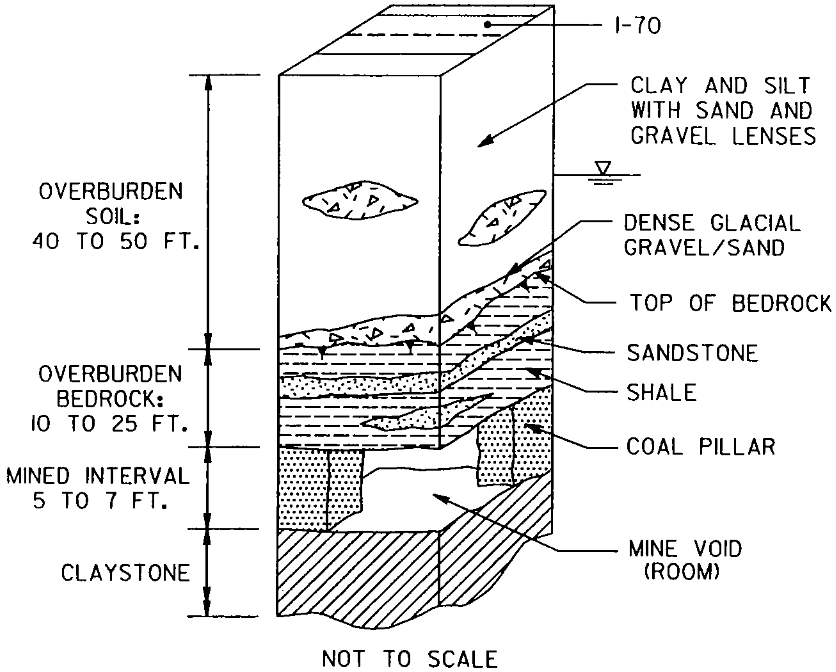


FIGURE 3: GENERALIZED SUBSURFACE PROFILE

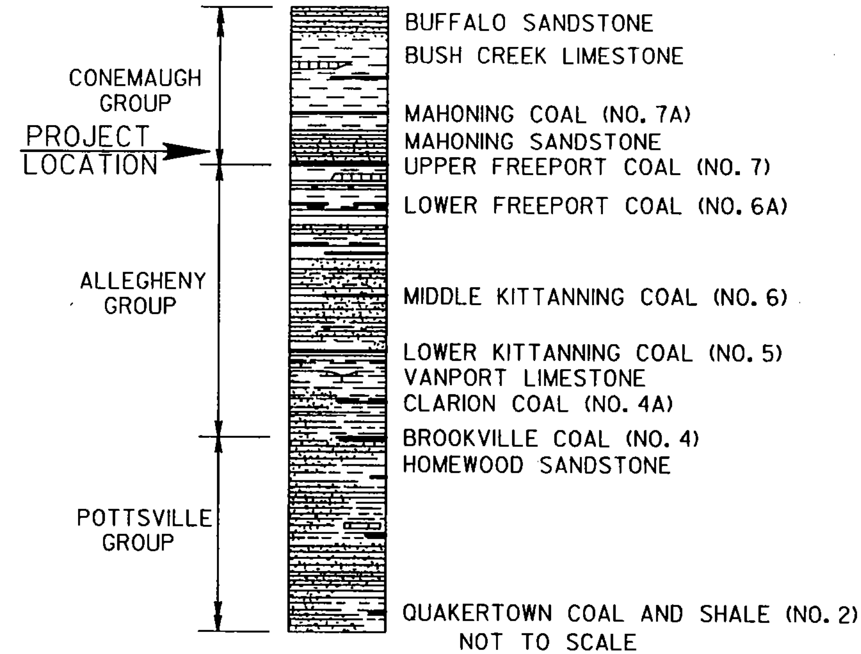
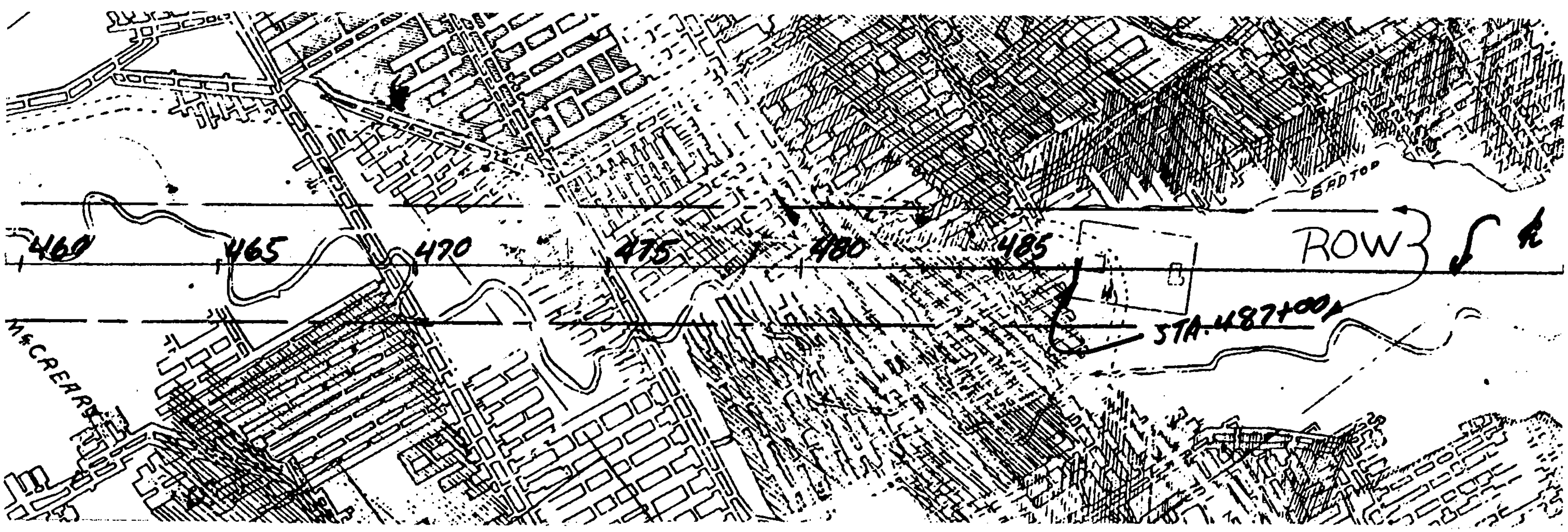


FIGURE 4: STRATIGRAPHIC COLUMN

FIGURE 5: MURRAY HILL NO. 2 MINE MAP



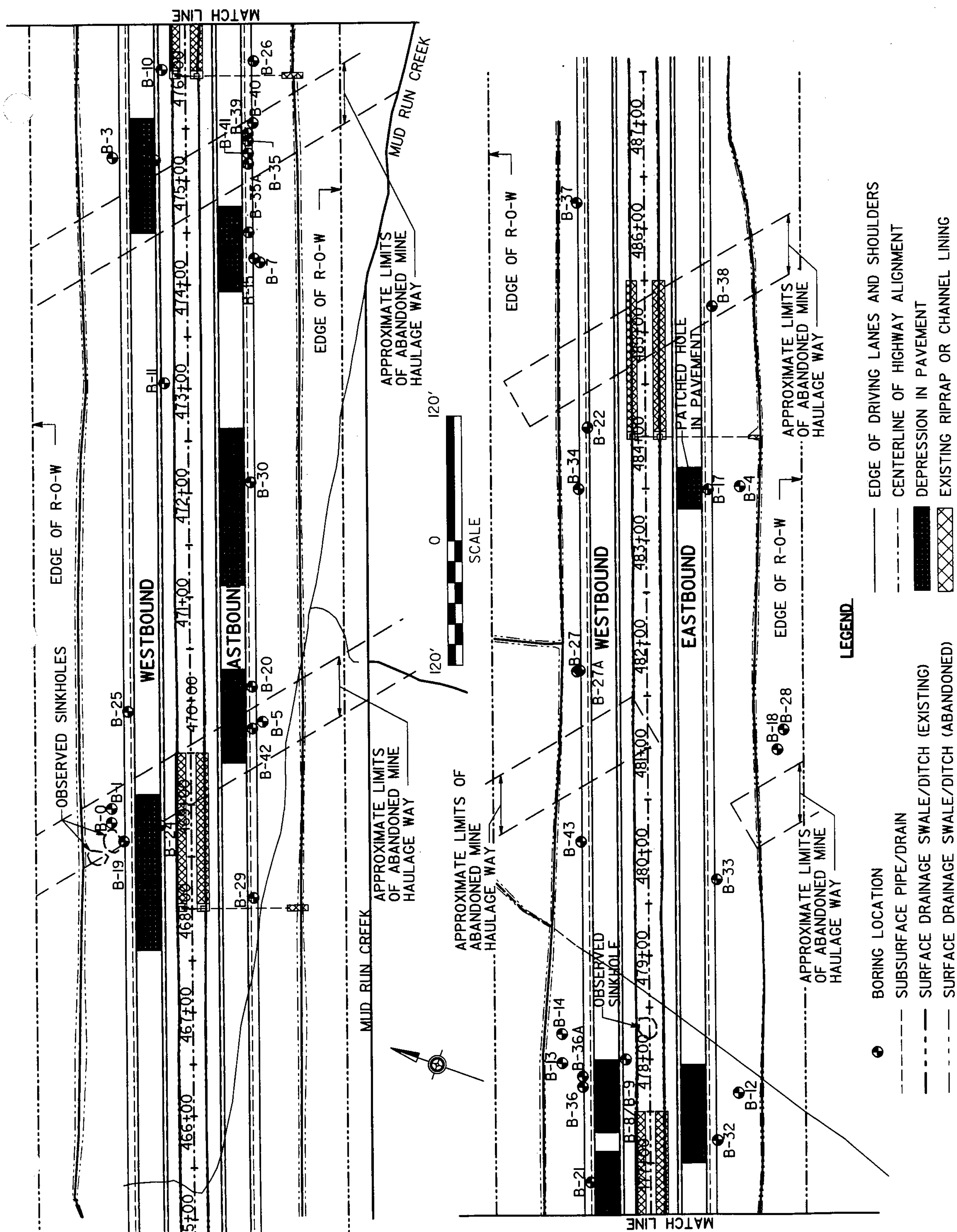


FIGURE 6: SITE PLAN

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typical room and pillar mines. Refer to Figure 5. However, the mine was not developed as most room and pillar mines are. Because of the inconsistent nature of the Upper Freeport Coal in this area, some entries were cut at odd angles and a few rooms were advanced in directions not typical to total extraction mining.

Mining at the site has occurred approximately between stations 468+00 and 487+00. Up to four entries were advanced through this area. Two of the entries, located near stations 469+00 and 475+00, are main passageways connecting two large sections of the mine. The two entries below the eastern end of the site, near stations 481+00 and 485+00, are not contiguous and appear to have been pushed in from each side of the valley specifically to access rooms.

It is apparent from the mapping that much of this area was prone to roof falls during mining. Parts of the map are labeled "bad top" near the edge of mining. In addition, large blocks of thick, good quality coal were left in place in the valley. It seems likely that the usual mining patterns were used to protect the existing haulage ways, keep the mine open in this area and prevent the mine from flooding.

__Surface Mining History__ Surface mining, including auger mining, was recently performed adjacent to the site next to workings of the King's Mine. It has been reported by the mine operator and ODOT personnel that abandoned workings were encountered resulting in prolific water flows into the mine's excavation that required constant pumping. The recent subsidence problems along Interstate 70 began shortly after pumping at the surface mine was started. Coal removal at the mine and the subsequent excavation dewatering have ceased, and the groundwater and mine water levels have returned to the pre-pumped elevations, about 20 feet above the mine roof.

PROJECT HISTORY

__Highway__ The subject section of Interstate 70 was designed and constructed in 1961/1962. The construction through the project area included the placement of minor fills of up to eight feet in thickness. Mud Run, flowing to the west, was routed into a rock lined drainage swale (ditch) located south of the traveled lanes. Flow from a tributary stream originating north of the new highway was routed into a rock lined drainage swale located north of the traveled lanes. Drainage swales on both sides of the highway are poorly graded, sloping less than one percent in many areas. A site plan is presented as Figure 6.

The original pavement section of the traveled lanes consisted of reinforced Portland Cement concrete pavement with bituminous concrete shoulders. The highway had a system of pipe underdrains located several feet below finished grade, running parallel to the traveled lanes. In 1989, the original concrete pavement was rubblized and a bituminous overlay was placed. Additionally, shallow underdrains, running parallel with the original underdrains, were installed.

__Subsidence Activity and Preliminary Geotechnical Investigations__ In undermined areas with relatively thin roof rock, a mine roof collapse can propagate upward without being stemmed by bulking of roof fall materials. Once the collapse reaches the relatively weak overburden soils, the void continues to propagate upward toward the ground surface. This type of pothole subsidence is typically localized and immediate in nature, producing deep, small diameter (less than 20 feet) sinkholes. Adjacent overburden and coal pillars are usually left intact, and the sinkholes typically correspond to significant mine features, such as main entries or haulage ways and intersections of rooms, pillars and haulage ways. In March of 1994, a pothole type subsidence feature was observed within the grassy median of Interstate 70. The hole measured approximately 10 feet in diameter and 10 feet in depth and was located near station 478+25. The hole was immediately filled with rock materials by Ohio Department of Transportation (ODOT) personnel. In April of 1994, ODOT contracted the drilling of four unsampled borings near the subsidence feature.

In September, 1994, a subsidence feature of similar size was observed in the westbound outside shoulder near station 468+50. This subsidence feature extended beneath the westbound traveled lane and caused the failure of the 6- inch underdrain. In addition to the subsidence feature, several depressions which had developed in the pavement, in both west and eastbound lanes, were noticed. Additional geotechnical investigations and physical site surveys were initiated. ODOT, the U.S. Department of the Interior, Office of Surface Mining Reclamation and Enforcement (OSMRE) and the Ohio Department of Natural Resources (ODNR) worked together to investigate the past deep mining and ongoing auger mining activities. ODOT advanced a total of nine new test borings; performed both

GROUND PENETRATING
RADAR (GPR)

ELECTROMAGNETIC CONDUCTIVITY
(EM)

SEISMIC REFRACTION
(SR)

FIGURE 7: GEOPHYSICAL TECHNIQUES

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conventional and infrared aerial photography; performed non contact profilometer surveys; and, performed detailed topographic surveys of the project area. ODOT also backfilled the shoulder failure with coarse aggregate and leveled the most severe pavement depression at station 483+50, eastbound with an asphalt patch. OSMRE personnel used the boreholes advanced by ODOT to view the overburden bedrock and the mined interval with a downhole video camera.

A total of 13 exploratory borings were advanced by ODOT at the project site as of October, 1994. Ten of the borings were not sampled and three were sampled using Standard Penetration Test (SPT) methods. Soils encountered in the borings ranged from 23 to 53 feet in thickness with most ranging from 40 to 50 feet thick. Soils were generally clays and silts with minor lenses of sands and gravels. Typical bedrock overburden thicknesses were from 10 to 25 feet with the rock type being mostly interbedded sandy shale and sandstone. Coal encountered in the borings was five to seven feet thick and was underlain by soft claystone. The bottom of the coal ranged from 64 to 80 feet below existing grade. Four of the borings encountered in place coal within the mined interval and seven of the borings encountered either voids or collapsed roof rock. The two remaining borings did not penetrate the mined interval.

ODOT initiated visual surveillance of the affected portion of Interstate 70. ODOT personnel visually inspected the pavement conditions at four hour intervals, 24 hours a day, seven days a week. The intent of the surveillance was to spot pavement depressions as they grew and to allow for the closure of the highway if pavement failures were observed.

GEOTECHNICAL AND GEOPHYSICAL INVESTIGATIONS

On October 4, 1994, Gannett Fleming Corddry & Carpenter (GF), of Columbus, Ohio, was requested by ODOT to participate in the project. GF's responsibilities included: technical consultation; design and implementation of subsurface investigations; identification of subsidence mechanisms; development of remedial design concepts; and preparation of final plans, specifications and cost estimates. Construction phase services were subsequently added to GF's scope.

The potential for failure of one of the traveled lanes due to deep mine subsidence was the primary concern of the project team. A two phased subsurface investigation program was developed. Phase One concentrated on the conditions of the near surface soils; most importantly, those soils supporting the traveled roadway. Phase Two concentrated on the exploration of the overburden bedrock and the mined interval.

Phase One Investigation It was theorized that mine roof collapses were propagating upward through the overburden bedrock and soil, ultimately leading to the observed pothole type failures. The primary purpose of the Phase One investigation was to probe the soils supporting the traveled lanes to look for voids working their way up towards the roadway. The Phase One program also was designed and executed to : collect soil type and strength information; determine top of bedrock elevations; attempt to identify conditions affecting subsidence potential such as soil type and soil stratigraphy, and to collect groundwater level data. In addition to roof collapse, it was theorized that movement of overburden soils into the mine interval was creating voids in soil and leading to subsidence events. It was suspected that groundwater was moving vertically downward due to a gradient created by the dewatering of the mined interval at the nearby coal augering operation. The downward movement of the water through the granular soil lenses identified by the ODOT borings, was suspected of carrying soil into the mine.

Borings A total of ten soil borings were advanced between October 20 and 24, 1994. Each boring was sampled using SPT methods and static water levels were measured in each boring. These borings did not penetrate bedrock. Generally the borings were positioned at or next to subsidence features and depressions, and at locations associated with subsurface anomalies identified by geophysical surveys. Borings were also positioned over main entries of the deep mine.

Geophysical Techniques Geophysical surveys were conducted to: detect and delineate voids in soil; determine the depth to bedrock; and, to distinguish between generally clayey and sandy subsurface soils beneath the roadway. It was anticipated that such a delineation would be helpful in limiting the required treatment area. GF's geophysics subconsultant, Enviroscan, Inc., of Lancaster, Pennsylvania, utilized ground penetrating radar (GPR), seismic refraction (SR) and electromagnetic terrain conductivity (EM) techniques to perform these surveys. Refer to Figure 7.

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Ground Penetrating Radar (GPR) represented the only method with a reasonable likelihood of success for detecting subsurface voids beneath the roadway. Electrical and electromagnetic terrain conductivity/resistivity techniques would have been ineffective for this project due to the presence of the reinforcing mesh and storm and underdrain piping. Seismic techniques exploiting the acoustic resonance of voids would have been difficult to interpret in the expected presence of considerable traffic-related vibration, and interference from high amplitude air blast and ground roll at depths of less than 30 feet.

GPR systems produce cross sectional images of subsurface features and layers by continuously emitting pulses of radar frequency energy from a scanning antenna as it is towed along a survey profile. The radar pulses are reflected by interfaces between materials with differing dielectric properties. The reflections return to the antenna and can be displayed on a video monitor as a continuous color cross section in real time, with a gradational color scale depicting the amplitudes of reflections as the magnitude of a particular subsurface dielectric contrast. On a color radar profile, the colors are mapped as the amplitude of the reflection (which is proportional to the magnitude of the dielectric contrast at the reflector). Therefore, color radar profiles contain information on the character of subsurface reflectors.

Since the electrical properties of air or water-filled voids are commonly different from most soils and backfill materials, they produce distinct and characteristic reflections. Air-filled voids are particularly distinct due to electromagnetic resonance or "ringing" or reverberation of the void reflection as repeated arrivals continuing vertically downward throughout the record.

In order to detect and delineate suspected potential voids at the Interstate 70 site, Enviroscan performed deep GPR scanning on the unreinforced roadway shoulders and medians using a monostotic 100 megaHertz (mHz) transducer capable of scanning to an estimated maximum depth of 20 to 30 feet. Since the presence of steel reinforcing in the roadway pavement prevented use of the penetrative but relatively low resolution 100 mHz transducer in the traffic lanes, a 500 mHz transducer with lesser penetration (6 to 8 feet maximum) but much better lateral resolution and the ability to penetrate the reinforcing grid was used to scan the traffic lanes. Data were displayed in real time on a color video monitor. The data were recorded on hi-8 video tape with verbal real-time field notes on the audio track.

The GPR field survey was performed on October 15, 1994 and included scanning of two continuous profiles along each of the four travel lanes and four shoulders using the 500 mHz transducer, and two profiles along each shoulder using the 100 mHz transducer.

Seismic refraction (SR) represented the most cost effective means of determining depth to bedrock since GPR could not be expected to penetrate to the necessary depths of up to 50 feet, and GPR and electrical or electromagnetic sounding techniques are difficult to calibrate for absolute depths in the presence of heterogeneous stratigraphy (i.e. fill and interbedded alluvium/colluvium).

Environscan completed a seismic refraction survey on October 28 and 29, 1994. The survey consisted of seismic refraction profiling along three lines - one on each grassy margin and one on the north edge of the median. Seismic refraction generally involves measuring the travel times of shock waves traveling from a surficial source (shot point) to a linear array of ground motion sensors (geophones). At a distance from the shot point, the first arrivals of seismic energy are waves that have been refracted along whatever seismic velocity (i.e. primarily density) contrast or contrasts are present in the subsurface, and the travel times of these arrivals can be used to compute a cross sectional profile of the seismic stratigraphy.

For the Interstate 70 survey, a Geometrics Smartseis S-12 seismograph was used to record seismic travel times at linear arrays or spreads of Mark Products 4.5 Hertz geophones spaced at constant 20 foot intervals along each of the lines. Travel times were recorded for shot points located at each end of each spread to provide multi-fold, reversed seismic data capable of resolving a potentially undulating bedrock surface.

At each shot point, seismic waves were generated by either 300 or 1,000 grain black powder charges set in 18 inch deep shot holes. Where traffic noise was detected in the seismic records, data from repeated charges were summed or stacked to enhance the signal-to-noise ratio.

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Processing and interpretation of the seismic refraction data were completed using the SIP package of computer programs developed for the U.S. Geological Survey by Rimrock Geophysical.

Electromagnetic Conductivity (EM) terrain conductivity mapping represented the most cost-effective means of delineating clay versus sandy subsurface soils since clay and sand generally have distinctly different electrical conductivities, but may have similar seismic velocities and gradational contacts or transitions indistinguishable using seismic or GPR methods.

Enviroscan completed an EM terrain conductivity survey of the Interstate 70 site on October 27, 1994 using a Geonics EM-31 instrument. The EM-31 was chosen since it can simultaneously record both subsurface electrical conductivity and an in-phase or metal detector response (for use in identifying areas where the terrain conductivity may be affected by the presence of subsurface or nearby or overhead metallic objects or structures. In addition, the EM-61 provides a good combination of penetration depth insensitivity to shallow interference from surficial debris or shallow, weather-related changes in soil conductivity, and vertical focussing to minimize interference from nearby metallic structures.

The EM survey was completed by collecting vertical dipole mode terrain conductivity and in-phase data at ten foot intervals along seven profiles. The lines were offset 15, 30, and 45 feet off of each outer edge of pavement, and along the center of the median. EM readings were collected by walking the EM-31 along the profiles and pausing to record readings at 10 foot intervals. Accurate stationing was maintained using a precision hip chain which was anchored and zeroed at the beginning of each profile.

In order to remove from the EM terrain conductivity data the effects of possible interference form the metal reinforcing in the roadway, the wire fences, underground metal drain pipes, buried metal debris, etc., the EM in-phase data were filtered to identify stations with a significant in-phase response indicating subsurface of nearby metal within the effective hemisphere of sensitivity of the instrument. These stations were removed from the data set to insure that the remaining terrain conductivities represented only the electrical properties of the subsurface soils.

The filtered terrain conductivity data were contoured using the statistical kriging routine in SURFER by Golden Software.

Results of Phase One Investigations

Borings The stratigraphy identified by the ODOT borings was generally confirmed by the Phase One borings. Additionally, the borings showed no voids were present in the overburden soils at the locations drilled at the time of the Phase One investigation. Static water level data showed that groundwater tended to be in either the upper 30 feet of the soil strata (above elevation 800 ±) or within 20 feet of the top of the coal (below elevation 780 ±). This demonstrated the presence of two water regimes at the site representing apparent perched water conditions in the overburden soils and flooded conditions in the mine.

Geophysical Techniques The GPR identified anomalies within it's effective penetration depth (20 to 30 ft) that were interpreted to be small voids or subsidence zones beneath the highway. Most appeared to be associated with previously identified sinks or depressions. Verification borings did not reveal subsurface voids at any anomaly locations. The GPR accurately identified the locations of the steel reinforcing mesh within the concrete pavement and the locations of cross pipes and underdrains.

The SR survey detected three velocity layers corresponding with roadway embankment materials, natural soils, and bedrock. The top of bedrock elevations determined by the SR were consistent with those measured in confirmation borings; generally corresponding within four feet. The SR also detected a constant low velocity zone along the top of bedrock over a 200 foot length of roadway. This was interpreted as bedrock that is more weathered or that has been fractured due to roof fall in the deep mines.

The EM survey showed an apparent clay content increase eastward across the site. The conductivities, however, were somewhat higher than expected for sand and clay soils and may have been affected by de-icing salt runoff.

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Phase Two Investigative Methods The goals of the Phase Two investigation included: sampling and classification of overburden soil, bedrock, and the mined interval; examination of the mined interval and the overburden bedrock using downhole video cameras; and verification of the SR top of bedrock profile.

Borings A total of 26 additional borings were advanced between November 28 and December 13, 1994. Each boring was sampled using SPT and NQ wireline, double tube, split inner barrel rock coring techniques. Static water levels were measured in each boring. The borings were advanced to investigate the following features: partially and fully extracted mine entries; partially and fully extracted room and pillar sections of the mine; apparent unmined areas; and surface deformations including sags, pavement depressions, and subsidence features.

Downhole Video Camera Borehole video logging and video logging of the mined interval was performed. The logging revealed that the overburden bedrock was fractured, but that large discontinuities occurred only within three feet of the roof of the mine. Above that level, the bedrock was very competent with only minor horizontal and infrequent high angle fracturing.

Results of Phase Two Investigations

Borings Soils encountered in the borings ranged in thickness from 23 to 53 feet with most ranging from 40 to 50 feet thick. Soils generally tended to be clays and silts, with minor interbedded lenses of sands and gravels. The upper 5 to 15 feet of soil below the highway consisted of soft to stiff clay and silt fill with rock fragments. Generally, below this layer, soft to stiff silt and clay with sand and gravel, up to 10 to 20 feet deep, was encountered. Blow counts in these upper layers averaged 8 to 12 blows per foot (bpf) with some as low as two or three bpf. Below these layers, a medium stiff to stiff layer of silt and clay with sand and gravel was often encountered down to the top of bedrock. This layer was noticeably varved (cyclic layering) and contained varying amounts of interbedded and intermixed sands and gravels. These varved sediments are typical of a lake environment. Most of the site, specifically the southwestern two-thirds, has a thin sand and gravel layer directly on top of bedrock. This layer, like most of the coarse sediments encountered in the borings, showed sub-rounded to sub-angular sediments composed of various rock types including sandstones, shales and coal. These sands and gravels were most likely randomly placed by either debris flows or ice rafts. Most of the interbedded gravel layers, especially the deeper ones, were very dense, almost having the appearance of glacial till. A 19 foot thick layer of this material was also encountered in boring B-38 at the northeast edge of the site. Some of the borings encountered a thin layer of residual soil.

Bedrock at the site consisted largely of sandy shale, shaley sandstone, coal and claystone. Bedrock overburden thicknesses above the coal ranged from nine feet at the western end of the site to 42 feet at the eastern end, with the rock thickening towards the east end of the site. However, typical bedrock overburden thicknesses were more in the range of 10 feet to 25 feet with the rock type mostly being interbedded sandy shale and sandstone. Overall, the bedrock was of good quality. Coal at the site was five to 7 feet thick and was underlain by soft claystone. The depth from the ground surface to the bottom of coal ranged from 65 to 80 feet across the site. In addition, the bottom of coal shows a sharp localized dip across the highway to the southeast of five to six degrees in places.

Water level observations were measured in all of the borings performed by GF and in the borings left open by ODOT. Water levels were collected immediately upon completion of the borehole and, when possible, a minimum of 24 hours after completion. As with the Phase One investigations, static water levels showed that a perched water condition exists in the overburden soils and that the mine interval is flooded.

Before construction of Interstate 70, all of the surface water runoff from the site flowed directly into Mud Run. Drainage from the ponds located north of the right-of-way drained to a channel that previously crossed the alignment. Now, since construction of Interstate 70, a large amount of surface flow from the watershed drains into the channel on the north side of the highway and is carried along the highway for up to a mile. Additionally, runoff from the highway itself is discharged into the swale to the south of the highway and is carried the same distance. These swales are poorly graded, sloping less than one percent in many areas. This poor drainage has produced soft, marshy areas in many places.

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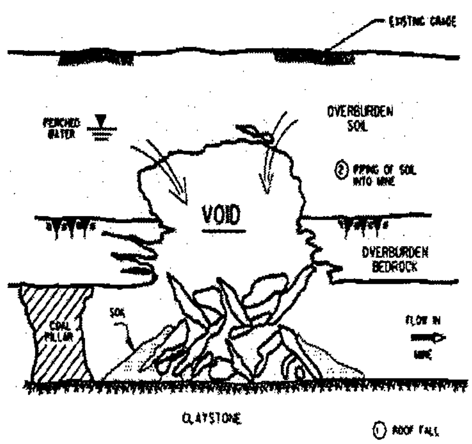
IDENTIFICATION OF SUBSIDENCE MECHANISMS

Based upon the data collected, two main mechanisms were identified as the most likely sources of the ground settlements, or subsidence features, observed at the project site. These mechanisms are mine roof failure and piping of soil into the mine void.

<u>Mine Roof Failures</u> Dewatering of the mined interval and the reduction of static water levels in overburden soils, due to the adjacent auger mining operations, is suspected to be the direct cause of the recent mine roof failures. In addition to the removal of buoyant forces on the overlying soils and bedrock, lateral restraining hydrostatic pressures were removed from the coal pillars. Combined with the weakening of the coal due to exposure to air, these changes in the state of stress of the coal pillars are believed to have lead directly to the failure of coal pillars, the mine roof and the overburden soils.

<u>Piping of Soils into the Mine Void</u> Phase One and Two boring data indicate that: the mine was flooded; overburden soils were saturated to approximately elevation 800 ±; and the bedrock was acting as an aquiclude. As the mine pool was drawn down by the auger mining operations and failures of roof rock occurred, a downward gradient between the soil and the mine interval was created. This downward flow of water within the overburden soils is believed to have carried soil into the mined interval, through the roof collapses. In addition, downward flow within subsidence features is expected due to the disturbed and potentially loose nature of the soils within subsidence features. Thus, subsidence features become larger and provide a mechanism for continued settlement. Figure 8 Illustrates the suspected mechanisms of subsidence.

FIGURE 8: SUBSIDENCE MECHANISMS



REMEDIAL ALTERNATIVES ANALYSES

ODOT and GF developed several remedial alternatives for consideration. The criteria for these alternatives required that the alternative eliminate the risk of catastrophic roadway failures and be constructable while maintaining two way traffic through the work zone on alternate sides of Interstate 70.

<u>Potential Treatment Intervals/Locations</u> Six potential locations at which some remedial treatment could be applied, either independently or in concert, were identified. These potential locations included:

- ***<u>The Adjacent Auger Mine:</u>*** To remove the catalyst of the mine roof failures and subsidence events, the permanent sealing of the adjacent auger mine was identified as a potential remedial alternative. In addition, the coordination of ODNR and ODOT in the review of future surface mining permit applications within the Murray Hill No. 2 Mine was recommended.

- ***<u>The Mined Interval:</u>*** To remove the possibility of future roof rock collapses into mine voids, the elimination of the voids was identified as a remedial alternative. This would be accomplished by filling the mine voids entirely with a bulk material, such as fly ash.

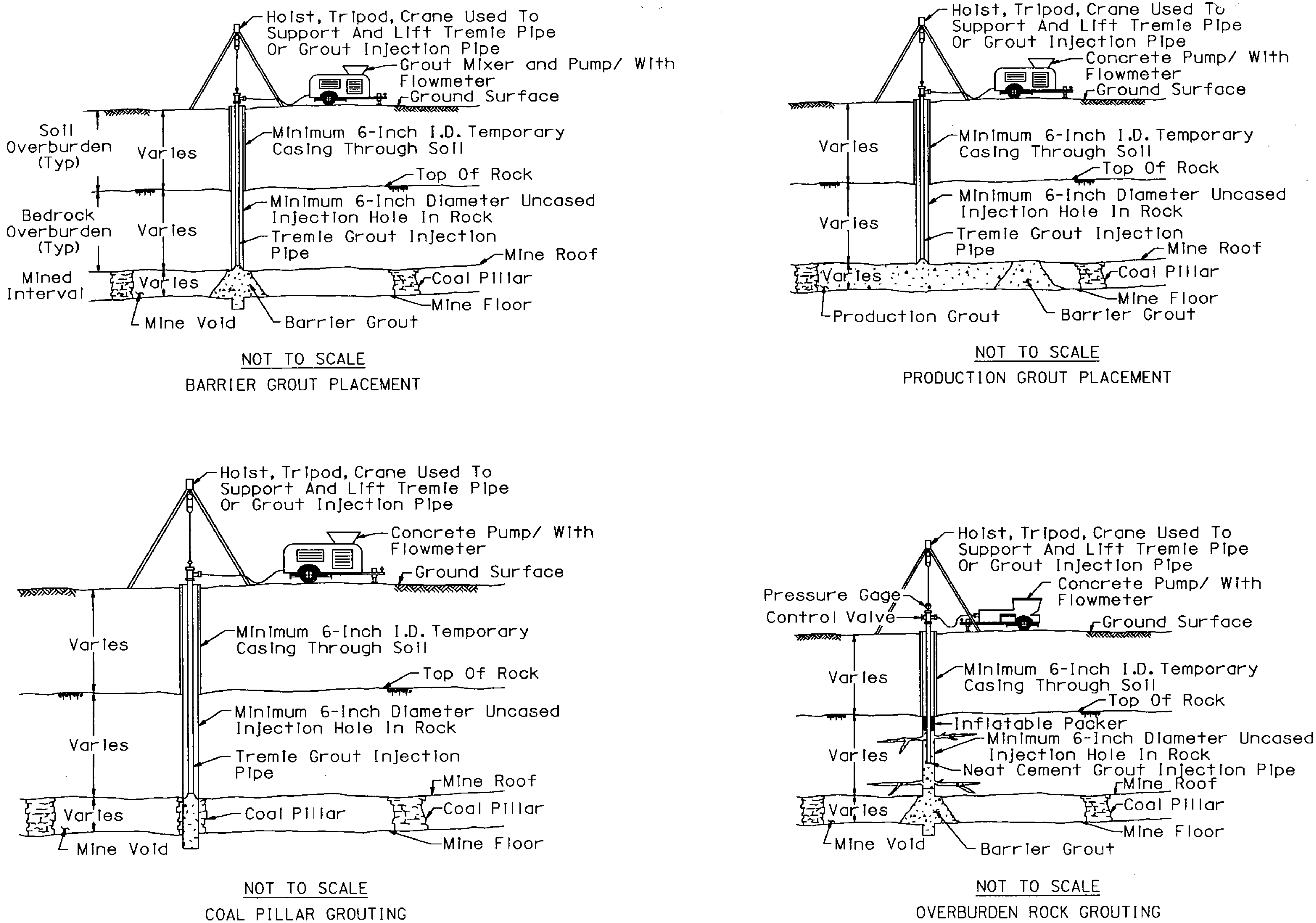


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FIGURE 9: REMEDIAL TREATMENTS

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- ***The Overburden Bedrock:*** Because the transport of soil downward through the bedrock was considered to be a potential subsidence mechanism, the consolidation grouting of joints, bedding planes and fractures within the bedrock was identified as a remedial alternative.

- ***The Overburden Soils:*** Soil improvement was considered as a remedial alternative. Methods considered included: compaction grouting; chemical grouting; jet grouting; and impact densification.

- ***The Highway:*** Another potential remedial alternative involved the excavation of the existing pavement section, median and drainage system to a depth of approximately six feet below grade; the placement of four layers of geogrid reinforcement ; and the replacement of the drainage system, median and pavement. This alternative would not reduce the likelihood of deep mine subsidence, but it would allow the highway to function by bridging, or spanning, subsidence features.

- ***The Drainage Swales:*** To reduce the infiltration of surface water into the overburden soils and, thereby, reduce the downward flow of water through the soils, the lining of the drainage swales adjacent to the highway with an impermeable clay liner was identified as a remedial alternative.

One other alternative was proposed by a general contractor. This alternative involved the total excavation of the highway, overburden soil, overburden bedrock, and the remaining pillars and mine gob and the replacement of the soils and rock and the construction of a new highway. This alternative option was ruled out due to the requirement to maintain limited traffic, the cost, the time required to perform the work and the fact that much of the excavated soil material would be too wet to be used as backfill, and the problems associated with dewatering the entire Murray Hill No. 2 mine.

Final Treatments Following an evaluation of cost and potential future risk, the following remedial alternatives were selected for final design (refer to Figure 9):

- Construction of a permanent seal at the adjacent auger mine and monitoring of future surface mining permits.
- Complete filling of the mined interval with a grout mixture consisting of fly ash, cement and sand.
- Consolidation grouting of the overburden bedrock.
- Tremie grouting of the overburden soil (boreholes and any voids encountered) with grout.
- Lining of the drainage swales with an impermeable liner.

REMEDIAL DESIGN FEATURES AND LIMITS

Treatment Limits Although the highway is underlain by abandoned mine workings over a distance of approximately 1,800 linear feet, final design limits included only the westernmost 1,200 linear feet of the undermined area. Additionally, the width of the treatment was limited such that only the traveled lanes and median were fully protected from subsidence. The treatment area was limited because: nearly all of the observed subsidence features and surface sags and depressions were located within the treatment area (with one notable exception); The borings, SR survey and mine maps showed that the bedrock was least competent and most fractured within the treatment limits; the borings revealed that overburden soils became more dense and thicker and the bedrock thickness increased towards the eastern end of the undermined area; and because surface water drainage conditions were much better at the eastern end of the area.

The pavement depression at station 483+50, eastbound, first noticed and patched by ODOT in September, 1994, was the only known area of distress located beyond the final design treatment limits. Final plans include a special treatment involving placement of barrier, production, consolidation and borehole grout for this area.

Treatment Descriptions

Auger Mine Seal The auger mine was reclaimed in February, 1995. At the time of reclamation, the mine pool water levels were significantly below final grades in the auger mined area and no surface flow was visible. Therefore, no

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seal was designed for the auger mine. ODOT and ODNR have arranged to monitor strip mine permit applications for the Murray Hill No. 2 mine in the project area.

Filling of Deep Mine With Grout The grouting of the deep mine will be accomplished by placing low strength fly ash, cement and sand grout in the mined interval. First, a barrier grout material will be placed within the mined interval around the perimeter of the treatment area. This barrier will serve to restrict the flow of a production grout material. The purpose of the production grout is to fill the existing mine void within the limits of the previously placed barrier. Both production and barrier grouts will be tremie injected, through drilled borings, under static pressures.

Consolidation Grouting of the Overburden Bedrock Upon completion of the barrier and production grouting operations, the overburden bedrock will be consolidation grouted under pressure using neat cement grout.

Tremie Grouting of Overburden Soils Boreholes in the overburden soils will be grouted under static heads with production grout materials. If voids are encountered during drilling, they will be filled during borehole grouting. The specifications allow for the pressure grouting of overburden soils as required.

Lining Drainage Swales The drainage swales adjacent to the highway will be lined with a Geocomposite Clay Liner. The grout mixes will be tested before use in the construction. A test construction program will be performed by the contractor. The test construction will involve the construction of a barrier, using the contractor proposed mix design and placement equipment, both on the ground surface and underwater. Production grout mixes will also be pumped. The purpose of the test is to evaluate the ability of the proposed grout mixes to build barriers or to flow freely, as required. Based upon the results of the test construction, proposed grout mix designs can be adjusted.

EPILOGUE

On Saturday, March 4, 1995, the area of patched pavement at eastbound station 483+50 failed catastrophically. Four vehicles hit the resulting hole, and one person was injured. The eastbound lanes of Interstate 70 was closed immediately by ODOT between the Cambridge and Old Washington exits. Traffic was detoured onto Interstate 77 and U.S. 40.

Because of the failure, the design team met in an emergency session. The purpose of the meeting was to: evaluate the appropriateness of the treatment limits; to review the expected effectiveness of the designed treatments and to evaluate the safety of the remaining portions of the undermined highway. Ultimately, ODOT found the entire undermined highway to be potentially unstable and closed the westbound lanes. Additionally, the treatment limits were extended to include the entire undermined length of the highway. ODOT is considering the placement of a continuously reinforced concrete pavement over the treated area upon completion of the remedial construction.

The plans, specifications and cost estimate were completed and the project was bid on Wednesday, March 22, 1994. The contract was awarded on March 23 and Nicholson Construction Company of Pittsburgh, PA, began operations on the site Friday, March 24, 1995. Construction is scheduled to last through June 21, 1995.

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Limestone Base Course Permeability

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Abstract

Base course permeability plays an important role in many pavement failures which are subjected to moisture-related problems. The permeability of the base course is dependent on the gradation of aggregates, particularly the amount of fines. This paper reports the change in permeability due to the variation of fines in a limestone base course.

A permeameter, obtained from the U.S. Bureau of Reclamation, was used to contain a 19 inch diameter by 9 inch thick limestone base course specimen. The 19 inch diameter permeameter was needed to accommodate a specimen with particles up to 1.5 inches in diameter.

The Arkansas Highway and Transportation Department (AHTD) Class-7 base course gradation was tested with 3%, 6.5 %, and 10% fines. The permeability of limestone base course material ranges from 5.52 x 10^{-3} cm/sec at 3% fines to 2.49 x 10^{-3} cm/sec at 10% fines. The decrease in permeability due to the increase in fines is 54.9%.

Introduction

A positive drainage system to remove free water from pavement structures is necessary in order for the pavement to have a long service life. The presence of excessive water in a paving system is known to be responsible for failures of both rigid pavement and flexible pavement. Water can cause premature rutting, cracking, faulting, increased roughness, and a decrease in the level of serviceability (Baldwin, 1987). Without a good drainage system, the pavement may suffer rapid deterioration under the action of pumping caused by dynamic traffic loading, and face the risk of frost damage.

Subsurface drainage design is a part of the pavement structural design procedure (Manual of Pavement Design Principles and Practices, 1987). Most pavement design procedures include some means of adjusting thickness or pavement life based upon the pavement drainage system. If drainage is poor, the base or wearing course thickness must be increased, resulting in a more costly pavement.

One of the best methods to evaluate the internal drainage of pavement is to measure the permeability of the least permeable layer. In a pavement, the least permeable layer is most often the base course.

The U.S. Bureau of Reclamation has developed a procedure for preparing coarse aggregate specimen like base course material and measuring the permeability (Earth Manual Part II, 1990). The USBR permeameter, which is a 19 inches diameter by 16 inches deep steel cylinder, is used to accommodate a specimen with particles up to 3 inches in diameter.

Permeability Testing

Permeability Apparatus

The permeability apparatus consist of the permeameter, the head tank, and the porous disk. The permeability apparatus was obtained from the Bureau of Reclamation, U.S. Department of The Interior. The *permeameter* (Figure 1) is a 19 inches diameter by 16 inches deep steel cylinder designed to contain a 9 inches thick base course specimen. The *head tank* (Figure 2) is a 6 inches diameter by 40 inches long plexiglas cylinder which contained water of 425.85 cm^3/in. The *porous disk* was placed at the inside bottom of the permeameter. The disk is made of a 19 inches diameter by 2 inches thick coarse grade carborundum porous material.

A $^1/_2$ inch thick closed cell medium density sponge rubber sheet was used as a liner for the inside wall of the permeability cylinder to prevent flow between the sample and cylinder wall. A rubber cement adhesive was used for attaching the sponge rubber liner to the inside wall of the permeameter (Figure 1).

A vibratory hammer (Figure 3), made by Wacker Corporation in Wisconsin (Model EHB 10/110), was modified and used for compacting the sample aggregates. The mechanical compactor was fitted with a 4 inches diameter compaction head. The Wacker was operated at 60 Hz frequency.

Materials

The limestone aggregates were obtained from McClinton Anchor quarry near West Fork. It is a bedded sedimentary deposit consisting mainly of calcium carbonate ($CaCO_3$). Gradation conformed to the Arkansas Highway and Transportation Department (AHTD) specification for Class-7 aggregate (Figure 4). Eighty five percent, by weight, of the fines were silt size particles. The fines by inspection were determined to be rock flour.

FIGURE 1. Permeameter and Closed Cell Liner.

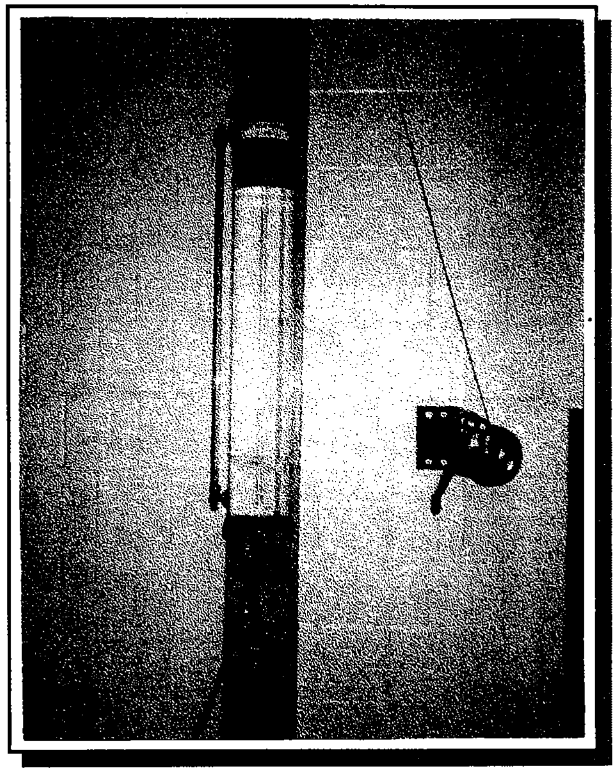

FIGURE 2. Head Tank.

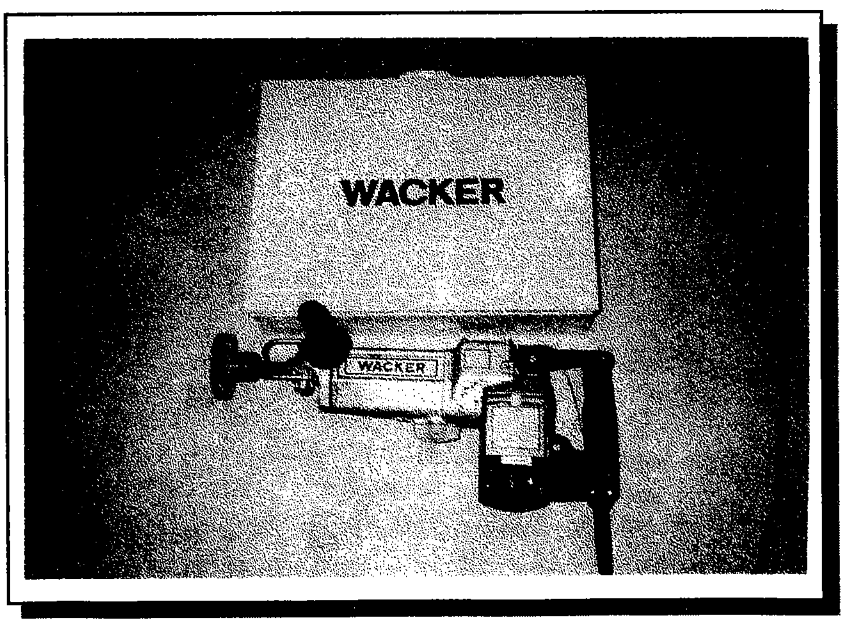

FIGURE 3. Wacker Mechanical Compactor.

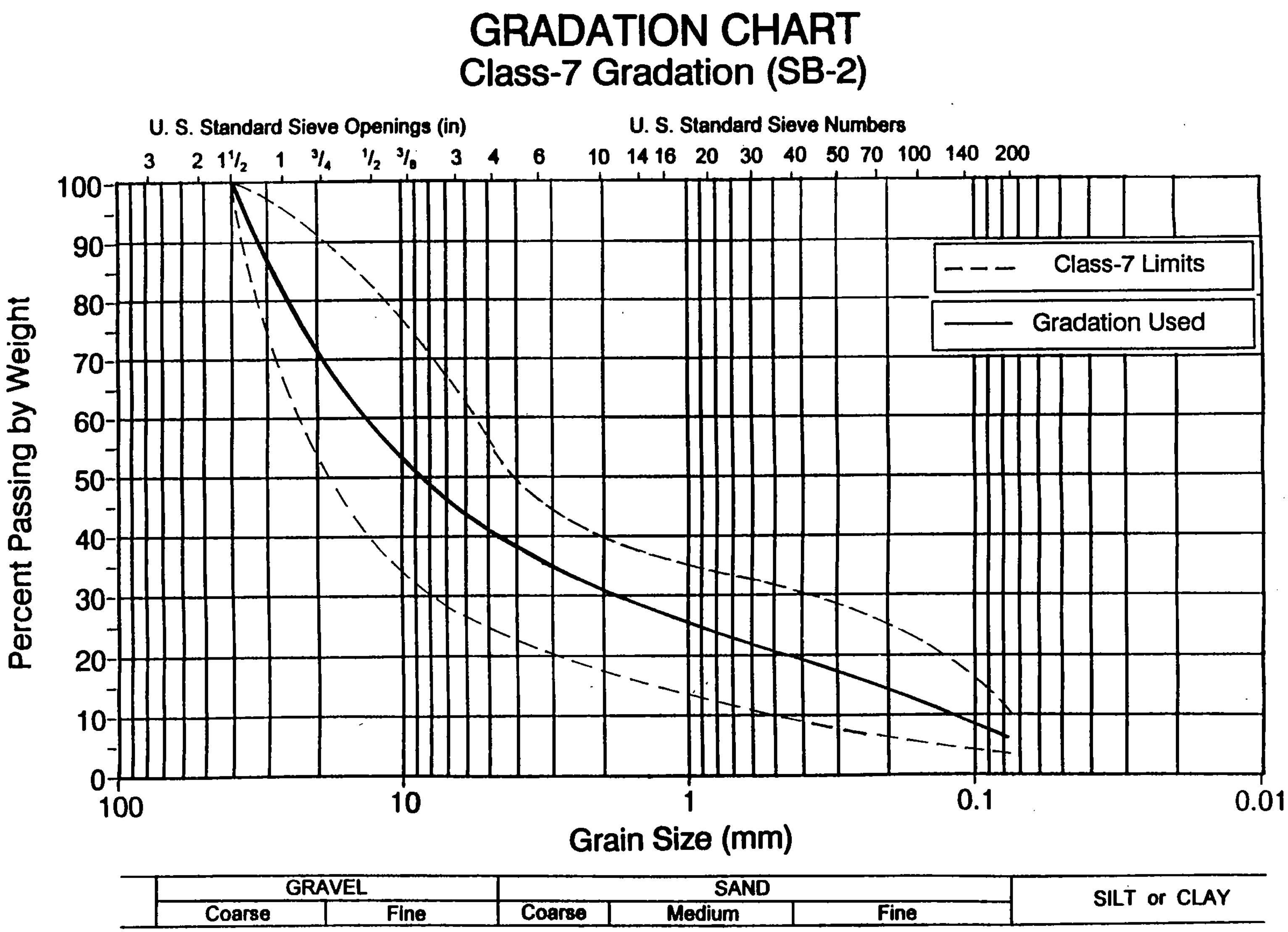

FIGURE 4. AHTD Class-7 Gradation and Grain Size Curves for Limestone Aggregate.

Sample Preparation

The base course specimens were built with three gradations in which the content of the fines (-#200) were varied from 3% to 10% by weight in order to find the effect of fines on the permeability. For construction of the 9 inch thick specimen, the sample was divided into three layers (Earth Manual Part II, 1990), each layer weighing 62 pounds. The compacted dry unit weight was 140 lb/ft^3

The addition of 4% water by weight was necessary in order to prevent segregation during the compaction process. Details of the testing and sample preparation are contained in the thesis "Permeability of Pavement Base Course" by Chin Leong Toh (1995).

Results and Analysis

Nine permeability tests were conducted on each specimen, three each at head differences of 3, 6, and 10 inches. Additional tests were conducted when the results were not consistent. The three best results for head differences of 3, 6, and 10 inches were averaged for all tests. The falling head permeability formula (Das, 1994) was used to calculate the coefficient of permeability.

The permeability for limestone is 5.52×10^{-3} cm/sec for 3% fines, 3.48×10^{-3} cm/sec for 6.5% fines, and 2.49×10^{-3} cm/sec for 10% fines. The decrease in permeability due to the increase in fines is 54.9%.

Permeability results and best fit straight lines, which were generated by using the least square best fit method, were plotted on a semi-log graph (Figure 5). The form of the semi-log equation of a best fit line is:

$$log_{10} k = -0.0486 P_{200} - 2.12$$

where, k = *permeability coefficient, cm/sec*
P_{200} = *percent fines used*

The existence of laminar flow was confirmed by comparing the ratio of selected flow rates to the ratio of selected hydraulic gradients obtained from the permeability test results. For a laminar flow, the flow rate in a saturated soil varies directly with the hydraulic gradient (Edward E. Johnson, Inc., 1966).

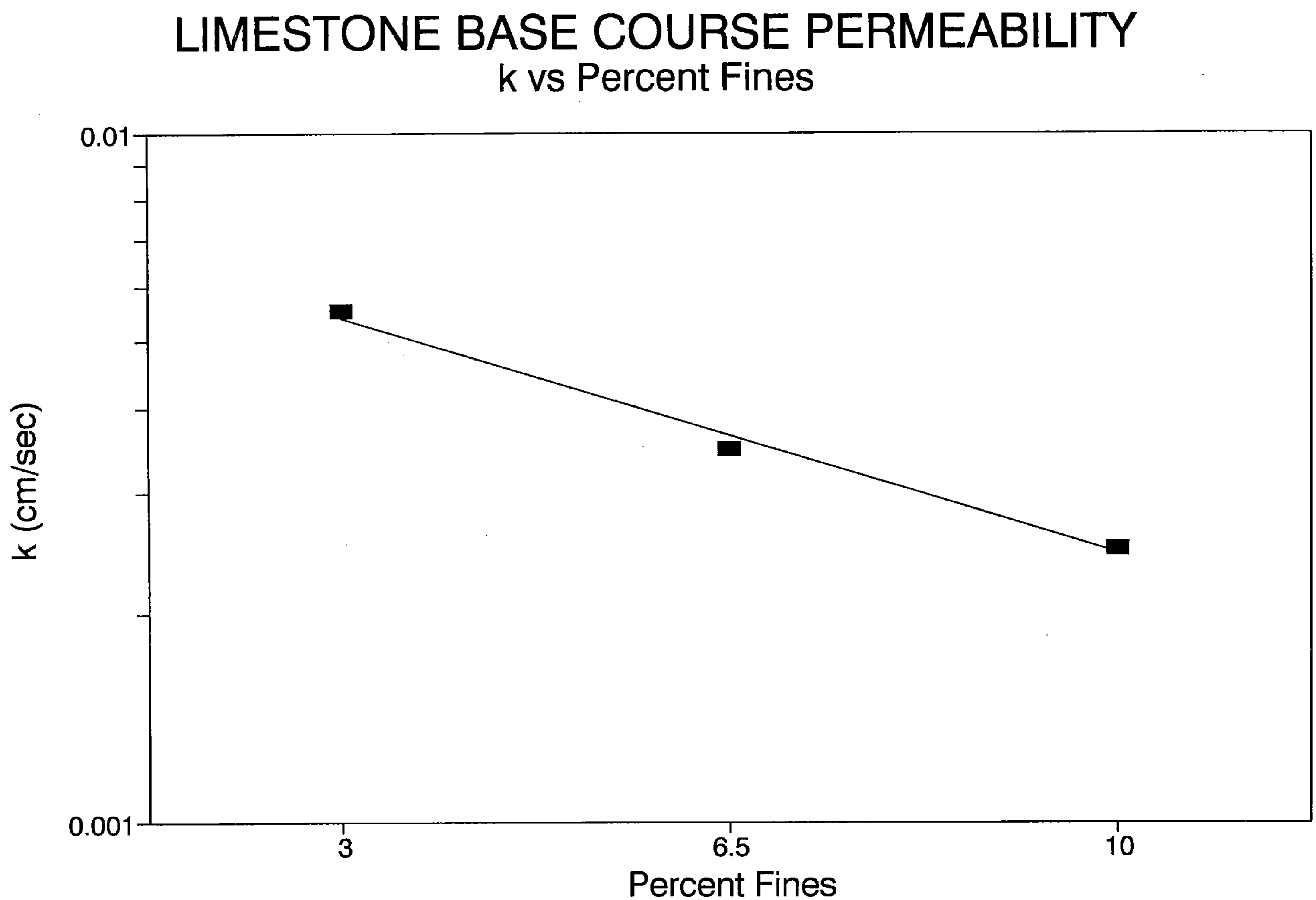

FIGURE 5. Semi-log Plot of Permeability Coefficient, *k* (cm/sec) vs Percent Fines for Limestone.

Conclusions

A laboratory procedure was developed for testing the base course permeability of a simulated field sample. The permeability test using the 19" permeameter was experimentally proven operational.

The permeability of limestone base course ranges from 5.52×10^{-3} cm/sec at 3% fines to 2.49×10^{-3} cm/sec at 10% fines. The decrease in permeability due to the increase in fines is 54.9%.

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USES OF SCRAP RUBBER TIRES IN CIVIL ENGINEERING

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ABSTRACT

Scrap rubber tires have received a great deal of attention, since improperly stored accumulations: attract insects and rodents, are a fire hazard, and are unsightly. Because tire rubber is a strong and durable material, reuse potentials in Civil Engineering are numerous. These include whole tires used in retaining walls, breakwaters, and submerged reefs. The sidewalls may be cut out and linked in mats to surface pioneer/construction roads or to form reinforcing layers in soil fill. Simple shredding of the tires produces a lightweight material for embankments and backfills. Reduction of the rubber to crumb size produces an additive which can improve the properties of asphaltic mixes. Pyrolysis of the scrap tires produces an impure carbon black, which has no established market, but may be usable as an asphalt additive.

The paper reviews all of the above applications, but emphasizes the uses of tire shreds, and the potential benefits of the carbon black resulting from pyrolysis of scrap tires on asphalt properties.

INTRODUCTION

If you are an average American, you contribute almost a tire per year to the previous accumulation of several billion. Chances are you pay a fee to a tire dealer to accept the old tire. This dealer typically accumulates these tires in a parked trailer belonging to a tire recycler, and pays the recycler to remove them. Since the tires cannot be stored outdoors, the recycler hauls them to a warehouse, where the tires which are reusable as tires are separated from those which are not.

Since the latter tires must either be stored indoors or hauled to another location, and the whole tires occupy too much space per unit weight to permit either operation to be efficient, the tires are generally shredded. At this point the retailer must either identify a market for the shreds or pay the haulage and tipping fee for disposal in a solid waste landfill.

Fortunately, there are many uses for scrap tires which are available in many forms: the whole tire, linkages of sidewalls cut from the tires, tire shreds, and crumbs ground from the tire rubber. Tire rubber is presumedly a renewable resource, so the reuse of the tire would seem to be less urgent than for other materials. However, improper storage of scrap tires has earned them a high priority for reuse. Such improper storage has produced fire, rodent and insect breeding hazards, and is unsightly.

Reuse should probably first focus on simple processing and low-technology approaches, e.g., tire shreds. This has not always been the guiding principle, and more complicated strategies like incorporating crumb rubber in asphalt have received much early attention.

WHOLE TIRES & TIRE SIDEWALL USAGE

The minimum processing requirement is that for reuse of the whole tire. The California Department of Transportation has reported on uses of the tires in small retaining structures, shoulder reinforcement, and channel slope protection (Caltrans, 1989). The applications emphasized are of a rapid maintenance kind of response, and the configurations proposed are simplified, i.e., not the result of extensive analysis.

In addition, the sidewalls may be cut from the tire (bead included) and linked into mats, which may be used as surfaces for pioneer or construction roads, or for reinforcing layers in soil fills. Both Caltrans and French experience are available. In addition, the Cold Regions Research and Engineering Laboratory (CRREL) of the Corps of Engineers is currently engaged in a field test at Fort McCoy, Wisconsin (Shoop, 1994). In this test a commercial variety of sidewall/tread linkage is being tested as a trafficability aid across thawed soils.

TIRE SHREDS

As mentioned previously, scrap tires are often shredded for convenience in storage and handling, as whole tires are not accepted for disposal in municipal solid waste landfills.

The tire shredder is a mill of cutting blades that is able to cut/tear through the structure of ordinary automobile tires, producing a collection of ragged handsized shreds. Some wire reinforcement may be pulled out of the shreds, but much more simply protrudes from the shreds. Successive feeds through the shredder reduce the shred size and pull out more of the steel wire. Shredders tend to be mobile. They may be purchased or "home-made", depending upon the financial resources of the recycler.

Three states come to mind with respect to research on the properties of a collection of tire shreds, viz., Wisconsin, Maine, and Indiana.

Most of the work cited herein has been accomplished/is being accomplished at Purdue University. An exceedingly talented graduate student, Imtiaz Ahmed studied the use of tire shreds in lightweight fill (Ahmed, 1993), and produced very encouraging results. Since he found that deformations of tire shred masses under load could be excessive, shred-soil masses were studied as well.

Tire shreds do not compact well, producing a unit weight of about 40 pcf with almost any application of compactive effort. The tire shred surface receives traffic loads rather poorly, deflecting considerably and perhaps providing punctures of rubber tires due to protruding wire.

The shred mass deforms considerably under sustained load due to a variety of mechanisms, including shifting of the shreds, straightening of curvature in the shreds, and elastic compression of the shreds. The latter two components are recoverable under unloading, which means that the mass flexes considerably under traffic.

Shearing resistance of the shred mass increases with strain. Values of c and ϕ are shown in Table 1. The hydraulic conductivity is high, comparable to gravel, which means that the shreds could be used in a confined drainage layer.

How are properties of the tire shreds changed when mixed with soil? The primary function of the soil is to fill the voids between the tire shreds, thereby increasing the shear strength and reducing the compressibility. The best mix is 40% tire shreds - 60% sand by weight. The strength is increased by addition of the sand, as well as the unit weight, which goes to about 80 pcf. See Table 1 for increased c and ϕ values.

The placement of tire shreds alone seems to produce no particular problems. In a similar way, use of shred-sand mixtures can probably be handled by placement of a layer of tire chips, followed by a layer of sand (in appropriate ratio), followed by equipment which imparts a significant amount of vibration. Once the sand has been vibrated into the underlying shred layer, the process is repeated.

While it seems advisable not to apply wheel loads directly to the tire shred mass, there is the possibility that this material would comprise an adequate subgrade layer for an overlying pavement. Both CBR and resilient modulus data were generated to address this point. The tire shred masses do not rate highly, but the shred-sand mixes seem adequate. A general rule of thumb would be to place about 3 feet of subbase/base/upper layer pavement between the wheel load and the shreds. When Falling Weight Deflectometer (FWD) tests are run on pavement over tire shreds it is found that indeed the deflection basin is deeper (than for compacted soil subgrade), but it is also broader, and has a lesser slope (Humphrey, 1995). This implies that fatigue stresses might

Material	Dry Density (pcf)	Strength Intercept c (psi)	ϕ (°)	Hydraulic Conductivity (cm/s)	Compressibility	References
Poorly graded clean sand, sand-gravel mix (SP)	100-120	-	37	10-100	-	Hunt (1986)
Medium sand, angular: Loose Dense	- -	- -	32-34 44-46	10^{-1}-10^{-2}	-	Leonards (1962)
Inorganic silt and clay, compacted (CL-ML)	100-120	9.4	32	10^{-4}-10^{-6}	-	Peck et al (1974)
2-in. tire chips in 6-in. diam. triaxial compression test	38	3.74	21	6-15	-	Bressette (1984)
1-in. chips, Std. Proctor Strain = 10% Strain = 15% Strain = 20%	40	 3.2 4.0 4.8	 14.6 20.3 25.3	0.6	 CR = 0.169 RR = 0.139 SR = 0.108	Authors
Ottawa Sand 1-in. chips 39%, sand 61% Strain = 5% Strain = 10% Strain = 15%	115 89	- 7.3 6.3 6.2	41.4 25.5 34.6 38.1	$1.6x10^{-4}$ $8.7x10^{-3}$	CR = 0.005 RR = 0.003 SR = 0.003 CR = 0.029 RR = 0.019 SR = 0.021	Authors
Crosby till, strain = 10% 1-in. chips 40%, till 60% Strain = 5% Strain = 10% Strain = 15%	119 81	10.7 5.6 8.2 9.4	29.4 12.7 21.1 27.0	$8.8x10^{-7}$ $8.8x10^{-3}$	CR = 0.030 RR = 0.007 SR = 0.003 CR = 0.096 RR = 0.056 SR = 0.045	Authors

Notes: 1. Compressibility parameters were determined for samples prepared using Standard Proctor compactive effort.

CR = Compression ratio, average slope of ε_v vs log (σ_v) in virgin compression zone (σ_v between 4 and 10 psi)

RR = Recompression ratio, average slope of recompression part of curve (σ_v between 4 and 10 psi)

SR = Swelling ratio, slope of rebound of curve (σ_v between 10 and 1 psi)

Table 1 Engineering properties of conventional fills and rubber soils (From: Bernal et al. 1993) be less harmful to an overlying pavement than would otherwise be believed.

Tire shreds and shred-sand mixes are also a good possibility for lightweight backfill material (Bernal et al., 1995). Such material is easily drained, and is lightweight. Experimental laboratory determination of the coefficient of lateral earth pressure is under way.

Using the shreds in a mechanically stabilized scenario is another promising use of them. Use of geogrids of a large opening size would seem to be particularly interesting, since the shreds would fit in between the openings in the geogrid. Pullout testing is underway to quantify the effects of geosynthetics of various kinds.

TIRE CRUMBS

Reduction of the scrap tire rubber to crumb size permits incorporation of them in asphaltic mixtures. In one process (the "dry" process) the crumb rubber serves as a small aggregate and the product is called "rubber-modified" asphalt. In another, the small crumb rubber is blended more thoroughly with the asphalt (the "wet" process) and the product is termed "rubber-asphalt".

The Intermodal Surface Transportation Efficiency Act (ISTEA) chose to target the use of crumb rubber in asphalt as the primary mechanism for disposing of scrap tires. This seems to be a questionable decision for a number of reasons: (1) it is costly to produce crumbs; (2) the effects of the crumbs on the parameters/ performance of the mix are complex and imperfectly understood; (3) experiences of state DOTs with the use of crumb rubber in asphalt is very mixed, and apparently is quite sensitive to the climatic and service environment; and (4) there are other (and more simple) ways of disposing of large numbers of scrap tires.

PYROLYSIS OF SCRAP TIRES

In the pyrolysis process the scrap tire is broken down into its basic components by the action of pressure and temperature. The products are mostly marketable: oil, gas, steel, and fiber. Fine sooty residual, an impure carbon black, is also produced, and while this material may be used in plastics manufacture, other uses may be desirable. Purer grades of carbon black have been studied as an asphalt additive by a number of researchers. In at least two states, Connecticut and Minnesota (Park 1995), the process has been carried forward to the road test stage.

In a recent study at Purdue, the impure carbon black produced at a pyrolysis plant in Indiana has been evaluated in the laboratory. Some very impressive improvements in hot mix asphalt parameters have been observed (Park 1995) when 15 to 20% of additive (by weight of asphalt) have been used. Study to determine whether the mix improvements justify the extra cost of the additive and its incorporation are still under way.

SUMMARY AND CONCLUSIONS

Accumulations of scrap tires have become an environmental problem in the U.S. Several billion scrap tires have been accumulated to date, and this number is increased every year at the rate of one-quarter billion per year.

Fortunately, there are viable options to the cutting/shredding of these tires and disposing in a solid waste landfill. Whole tires may be stacked and used to retain soil slopes, or as breakwaters, or as submerged reefs for marine life. Alternatively, the sidewalls may be cut and linked into mats that surface pioneer or construction roads, or form reinforcing layers in soil fill.

Perhaps the use that can involve the largest tire quantities with the most simple processing is that of shreds to form embankments or backfills. When the shreds are used alone, the resulting unit weight is only one third that of natural soils. Some improvement in geotechnical parameters can be achieved by mixing the shreds with sand. The resulting unit weight is still only two thirds that of natural soil.

Finally, reduction to crumb size permits incorporation as an additive in asphalt. Another possibility is the incorporation of the impure carbon black from pyrolysis of scrap tires as an asphalt additive.

Tire rubber is a strong and durable product which is too valuable to be treated as a waste. It burns, but its use does not appear to produce any important environmental problems.

ACKNOWLEDGEMENTS

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Evolution of a Technique: Petrography of Aggregates for Concrete and Bituminous Highway Pavements

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ABSTRACT

Petrography has been used to evaluate concrete aggregates for the past 60 years dating back to work initiated by Mielenz and his coworkers. Texture and composition of rocks are viewed megascopically and microscopically using polished sections and thin sections to make critical determinations. These data are augmented by results from standard highway engineering tests and laboratory analyses including x-ray diffraction and scanning electron microscopy. Problems of concrete deterioration evaluated using petrography include alkali silica reaction, alkali carbonate reaction, D-cracking and sulfate attack, among others.

Aggregate petrography is applied to bituminous pavements to evaluate frictional resistance of surface courses. Polishing of the coarse aggregate by vehicular traffic leads to reduced frictional resistance and poorer skid resistance. Petrographic evaluation in a recent Indiana study has identified those aggregate types which provide the greatest frictional resistance for bituminous overlays for high traffic highways. Blast furnace slag, crushed glacial gravels, quartz sandstone and certain reef dolomites were shown to provide higher frictional resistance. Petrographic details were useful in delineating the characteristics of these aggregates.

Additional problems persist regarding aggregates for bituminous and concrete pavements. Concrete deterioration from the formation of ettringite is now suspected in pavements in Iowa and the deterioration of bituminous aggregates by de-icing salts is under consideration in Indiana. Petrographic studies provide an integral part in both of these evaluations. Petrography of aggregates is a specialized study involving geology and highway engineering. Purdue University is one of the few locations in the United States where a graduate course in this subject is available for study.

Introduction

Petrographic examination of aggregates for concrete and bituminous pavements involves the visual examination of the texture, mineralogy and properties of individual particles that comprise the samples. This requires the use of a hand lens for megascopic examination plus petrographic and stereographic microscopes for higher magnification. In some cases, x-ray diffraction, differential thermal analysis and election microscopy are used in various combinations to augment the optical microscopy. By this overall procedure, the relative abundance of specific types of minerals and rocks is determined, along with their physical and chemical properties including particle shape, surface texture, pore characteristics, hardness, and potential chemical reactivity. Also mineral coatings and the presence of contaminating substances are identified, and evaluated in terms of the proposed conditions of service in the pavement.

Petrographic evaluation of aggregates has been in use since 1936 by the Bureau of Reclamation, U.S. Department of Interior and similarly by the U.S. Army Corps of Engineers since before 1940. An extensive bibliography on petrographic evaluation is presented by Mielenz, 1994, in Petrographic Evaluation of Concrete Aggregates, ASTM, STP 169C. Aggregates for portland-cement concrete or bituminous pavements are also evaluated using petrographic methods by the Ontario, Ministry of Transportation (Rogers, C.A., 1990) and by various agencies of the U.S. government and the state departments of transportation.

In 1952, ASTM C 295, Recommended Practice for Petrographic Examination of Aggregates for Concrete was accepted, and subsequently adopted as a standard in 1954. It appears in a somewhat modified form in the current Annual Book of ASTM Standards (C 295) and is cited in the basic reference for aggregate specifications, ASTM C33.

Valuable information obtained in a reasonable amount of time, based on petrographic examination justifies this procedure in the investigation, selection, manufacture and use of aggregates for concrete and bituminous pavements. Petrographic evaluation can be applied to gravel, sand, crushed stone, blast-furnace slag, light weight aggregates, and recycled hardened concrete for use in construction.

Petrographic Examination

Petrographic examination of aggregates can be performed in the field as well as in the laboratory. Sand and gravel materials have been termed “natural aggregates” as contrasted to crushed stone or quarried stone derived from bedrock sources. Typically, only preliminary evaluation is accomplished in the field, followed by detailed laboratory analysis. Field sampling must be carefully performed to obtain representative samples. Cobble sized, natural aggregates (>3”) may require a detailed petrographic field evaluation because of the large size needed to provide a representative sample (several hundred pounds typically). If the deposit or bedrock exposure is highly variable, sampling is required from each zone and field notes provide a description of the relative proportions of materials present. The proportions of unsound, fractured or chemically deleterious materials should be estimated from field measurements.

Samples provided to the laboratory consist of 1) gravel, sand, crushed stone, slag or synthetic aggregate, 2) stone from a bedrock sources in quarried blocks, irregular pieces or drilled cores, 3) pieces or drilled cores of concrete which contain the aggregate of interest. For fully graded samples in 1) above, the examination should be performed on at least three coarse size fractions from the gradation. Each size fraction should consist of at least 150 pieces.

Fine fractions (such as the +No.30 sieve size) are best identified and particle types counted using the stereographic microscope. Typically five fine fractions are evaluated in addition to the three coarse fractions mentioned above. For the finer fractions, mineral composition is most accurately determined using immersion oils or thin section mounts using the petrographic microscopes.

Thin sections, occasionally employed to examine natural aggregates, are more typically used to study quarried (or crushed) stone. Thin sections or polished sections, supplemented by oil immersion mounts of granular material usually are used to analyze blast-furnace slag aggregates. For the finely divided or well dispersed constituents in natural aggregates or crushed stone, x-ray diffraction and differential thermal analysis may be required for identification purposes.

Details for calculating and reporting the results for the petrographic examination of aggregates are summarized on ASTM C 295. Regarding the analysis of aggregates enclosed in hardened concrete, the point-count analysis on polished surfaces is employed to determine the proportional volume of the total aggregate and of selected lithologic aggregate types.

An outline for the observations that should be considered during petrographic evaluation of aggregates for concrete and when applicable, for bituminous pavements is presented in Table 1. This is based on information provided by Mielenz, 1994. Problems of concrete deterioration evaluated using petrographic techniques include alkali silica reaction, alkali carbonate reaction, D-cracking, sulfate attack, carbonation, staining.

The condition of individual particles is determined during the petrographic examination. Physical condition is classified based on three categories: 1) satisfactory, 2) fair, and 3) poor. Chemical stability in concrete is designated as 1) innocuous, and 2) potentially deleterious. These categories are discussed in detail by Mielenz (1994).

Table 1. Observations to Consider in Petrographic Examination of Aggregates for Concrete and Other Materials.

- Mineralogic and lithologic composition
- Particle shape
- Surface texture
- Internal fracturing
- Coatings
- Porosity, permeability, and absorption
- Volume change, softening, and disintegration with wetting-drying
- Thermal properties
- Strength and elasticity
- Density
- Hardness
- Chemical activity
    - Solubility
    - Oxidation
    - Hydration
    - Carbonation
    - Alkali-silica reactivity
    - Alkali-carbonate reactivity
    - Sulfate attack on cementitious matrix
    - Staining
    - Cation-exchange reactions
    - Reactions of organic substances
    - Effects of contaminants

Poor particles, those non desirable constituents in concrete, possess one or more of the following characteristics: friable to soft; slake when wetted and dried; highly fractured; high capillary absorption, or significant volume change during wetting and drying. Potentially deleterious particles are those which in portland-cement concrete yield significant expansion, interfere with cement hydration or produce other harmful effects on the concrete. Aggregates, petrography and concrete are discussed in the recent text by T.R. West (1995), Chapter 6.

An example petrographic evaluation is provided in Table 2 for a glacial gravel in northern Indiana, examined by T.R. West. The aggregates are grouped according to physical condition, the chemical condition is indicated in the two columns. A total of 12.9% potentially deleterious material is indicated.

Table 2. Petrographic Evaluation.
Natural Aggregate, Northern Indiana, 1/2"-3/8" Fraction.

Satisfactory	Wt%	Innocuous	Deleterious
Limestone (hard)	24.3	x	
Dolomite (hard)	16.6	x	
Limestone (slightly weathered)	2.1	x	
Dolomite (slightly weathered)	1.9	x	
Limestone (sandy, hard)	0.6	x	
Dolomite (sandy, hard)	1.0	x	
Sandstone (hard)	1.1	x	
Sandstone (medium hard)	0.6	x	
Gneiss (hard)	1.2	x	
Quartzite (hard)	6.8	x	
Graywacke-arkose (hard)	1.5	x	
Rhyolite-andesite (hard)	1.2		x
Granite (hard)	13.9	x	
Diorite (hard)	9.1	x	
Basalt (hard)	1.3	x	
Chert (slightly weathered)	4.7		x
TOTAL SATISFACTORY	87.9		
Fair			
Limestone (moderately weathered)	0.1	x	
Dolomite (moderately weathered)	0.4	x	
Gneiss & schist (moderately weathered)	0.4	x	
Granite (moderately weathered)	1.1	x	
Diorite (moderately weathered)	0.4	x	
Chert (moderately weathered)	3.1		x
TOTAL FAIR	5.5		
Poor			
Sandstone (soft and friable)	0.7	x	
Granite-diorite (friable)	0.5	x	
Chert (deeply weathered)	3.9		x
Ocher (deeply weathered)	1.1	x	
Shale	0.4	x	
TOTAL POOR	6.6		

A numerical evaluation for the megascopic evaluation of aggregates has been developed for use by the Ontario Ministry of Transportation. Dating back to 1948 in work by Bayne and Greenland (1948), aggregates for highways were evaluated according to the Petrographic Number, PN. Revisions in the evaluation procedure were made in 1954 and 1989 to provide a numerical system which is more sensitive to field performance of aggregates in highway pavements (Rogers, 1990).

Table 3 shows a Petrographic Number Analysis, using the 1989 procedure, Ontario Ministry of Transportation. Represented is the same aggregate evaluation provided in Table 2, a Northern Indiana gravel, 1/2 to 3/8" size fraction evaluated by T.R. West.

The 1989 PN evaluation is used in conjunction with an unconfined aggregate freeze-thaw test by the Ontario Ministry of Transportation to predict field performance of aggregates in highway pavements (Rogers, 1990). Refinements in the numerical evaluation were accomplished through related studies (Mielenz, 1958; Hudec, 1983) to yield the 1989 PN evaluation.

In Table 3, the PN for concrete and bituminous pavements value is 159.4. According to data from the Ontario Ministry of Transportation, (Rogers, 1990), 160 is about the maximum allowable PN for pavement aggregates under climatic conditions common to Ontario. As stated above, they also use the unconfined aggregate, freeze-thaw test as an acceptance criteria.

In Table 3 and adjusted PN for base courses is also shown. A lower factor is appropriate in some cases for base courses as compared to pavements for some hard particles which are less desirable constituents as pavement aggregates. The adjusted factor is a "give back" factor equal to 2 for fair aggregates, whereas under the poor category an adjusted factor of 5 for cherty aggregates and 3 for most other soft materials is applied. Note that shaley, clayey and ochorous carbonates receive no adjustment in the poor category and there are no adjustments for any deleterious aggregates. An adjusted PN for bases is 115.9 for this gravel sample.

The petrographic evaluation of aggregates for highway use has a long established history at Purdue University. Early work by Shupe and Lounsbury (1958) was followed by that of Schuster (1961) and Aughenbaugh et al. (1962). West, et al. (1966, 1970) provided petrographic evaluation in regard to base course degradation of pavements throughout the United States. In that study, natural aggregates and crushed stone were evaluated both megascopically and in thin section. Detailed statistical measurements were made in thin section including grain size, grain size distribution, matrix percent, void content specific surface, etc.

Results were presented in several publication (West et al. 1970, and West, 1969). A major conclusion of this study was that aggregates consisting of different rock types require

TABLE 3.

Petrographic Number Analysis - 1989 Procedure, Ontario Ministry of Transportation

Aggregate Source: Northern Indiana, 1/2"-3/8" Fraction

Rock Name	Wt.%	Factor Conc. & Bit Pave.	Weighted Value	Adj. Factor Bases	Weighted Adjust.
Limestone (Hard)	24.3	1	24.3		
Dolomite (Hard)	16.6	1	16.6		
Limestone (Slightly weathered)	2.1	1	2.1		
Dolomite (Slightly weathered)	1.9	1	1.9		
Limestone (Sandy, hard)	0.6	1	0.6		
Dolomite (Sandy, hard)	1.0	1	1.0		
Limestone (Sandy, medium hard)		1			
Dolomite (Sandy, medium hard)		1			
Limestone (Crystalline, hard)		1			
Dolomite (Crystalline, hard)		1			
Sandstone (Hard)	1.1	1	1.1		
Sandstone (Medium hard)	0.6	1	0.6		
Gneiss-Schist (Hard)	1.2	1	1.2		
Quartzite (Coarse and fine grained)	6.8	1	6.8		
Graywacke-Arkose (Hard and medium hard)	1.5	1	1.5		
Volcanic (Hard and slightly weathered)	1.2	1	1.2		
Granite-Diorite-Gabbro (Hard)	23.0	1	23.0		
Basalt (Hard)	1.3	1	1.3		
TOTAL GOOD AGGREGATE	83.2		83.2		
Limestone (Crystalline, slightly weathered)	0.1	3	0.3	2	0.2
Dolomite (Crystalline, slightly weathered)	0.4	3	1.2	2	0.8
Limestone (Soft, slightly shaly)		3		2	
Dolomite (Soft, slightly shaly)		3		2	
Limestone (Sandy, soft and soft pitted)		3		2	
Dolomite (Sandy, soft and soft pitted)		3		2	
Limestone (Deeply weathered)		3		2	
Dolomite (Deeply weathered)		3		2	
Gneiss (Soft), Schist (Medium hard)	0.4	3	1.2	2	0.8
Chert, cherty carbonates (Unweathered to slightly)	7.8	3	23.4	2	15.6
Granite-Diorite-Gabbro (Brittle)	1.5	3	4.5	2	3.0
Volcanic (Soft)		3		2	
Encrustation		3		2	
Argillite		3		2	
TOTAL FAIR AGGREGATES	10.2		30.6		
Limestone (Shaly or clayey)		6			
Dolomite (Shaly or clayey)		6			
Limestone (Ocherous)		6			
Dolomite (Ocherous)		6			
Chert, cherty carbonate (leached)	3.9	6	23.4	5	19.5
Sandstone (Soft and friable)	0.7	6	4.2	3	2.1
Volcanic (Very soft, porous)		6		3	
Limestone, Crystalline (Soft)		6		3	
Dolomite, Crystalline (Soft)		6		3	
Gneiss (Friable)		6		3	
Granite-Diorite-Gabbro (Friable)	0.5	6	3.0	3	1.5
Cementations		6		3	
Cementations (Total)		6		3	
Schist (Soft)		6		3	
Siltstone		6		3	
TOTAL POOR AGGREGATES	5.1		30.6		
Ocher	1.1	10	11.0		
Shale	0.4	10	4.0		
Clay		10			
Volcanic or schist (decomposed)		10			
TOTAL DELETERIOUS AGGREGATES	1.5		15.0		
TOTALS	100.0				43.5

% Good 83.2 x 1 = 83.2
% Fair 10.2 x 3 = 30.6
% Poor 5.1 x 6 = 30.6
% Del. 1.5 x 10 = 15.0
SUM = 159.4 Conc. & Bit Pavements
Corrected for Bases: 159.4 - 43.5 = 115.9

different testing procedures, both petrographic and standard highway types; to evaluate their performance regarding degradation.

Also at Purdue University in the 1980s several petrographic studies of highway aggregates were performed. A.Shakoor under T.R. West's direction studied carbonate aggregates used in bituminous wearing surfaces (Shakoor and West, 1979) and argillaceous carbonate aggregates for concrete pavements (Shakoor et al., 1982). In the latter study, deteriorating coarse aggregate lithologies were traced back to their source in specific quarry ledges. Clay-rich rocks were found to yield unsuitable particles that prevailed through the quarrying and production processes to yield unsuitable constituents.

Current Research at Purdue University

Currently, a 2-1/2 year study on the evaluation of Indiana aggregates for use in bituminous overlays is being completed at Purdue University. A progress report on this research was presented at the 1994 Highway Geology Symposium (Bruner, et al., 1994). In this study, aggregate petrography comprised an important part of the research effort.

After years of wear, bituminous overlays are placed over concrete pavements to re-establish a smooth riding surface. Frictional resistance of the bituminous pavement must provide the needed braking ability for vehicles, and the coarse aggregate is called upon to supply primary roughness.

Polishing of the coarse aggregates reducing frictional resistance is a common concern for bituminous pavements. In Indiana a combination of blast-furnace slag and dolomite has been used to increase frictional resistance. Limestones on average have performed poorer than have dolomites. In this study polish susceptibility was determined using the British Wheel and Pendulum testing equipment. By this method, a BPN, British Pendulum Number was obtained.

Under existing highway specifications, dolomites must contain 10.3% elemental magnesium to qualify as an acceptable aggregate for bituminous overlays. This corresponds to a minimum of 78.1% dolomite in the aggregate. Acid-insoluble residue contents of these carbonates are determined which consist primarily of quartz and clay.

Thin sections were examined for the carbonates to determine composition and texture. X-ray diffraction was used to provide quantitative information on the carbonate composition.

Grain size, grain size distribution void content and matrix to grain ratios were also obtained. Textural parameters were compared to BPN values.

Crushed river gravels were also considered as an aggregate source for bituminous overlays. Their performance was evaluated using the British Wheel and Pendulum Test. Petrographic evaluation of these rock types was also accomplished. The purpose of the research was to determine how well different aggregates resist polishing and to relate aggregate properties to frictional resistance. Standard laboratory test results were also considered, including sulfate soundness, absorption, Los Angeles abrasion and freeze-thaw loss. The results of the testing program are currently being evaluated and will soon be presented for review to the sponsoring agency.

Course Offering, Petrography of Aggregates

A course in petrography related to concrete aggregates was initiated at Purdue University in the late 1950s by R.W. Lounsbury, an igneous petrographer. Students taking geology courses at that time were mostly civil engineers so that the traditional thin section petrography course was adapted according to engineering applications. T.R. West, a student of Lounsbury's used these techniques in his Ph.D. research on aggregate degradation (West, 1966; West et al., 1970). Following the departure of R.W. Lounsbury from Purdue in the late 1960s, T.R. West modified the course to incorporate more detail on engineering aggregates plus additional emphasis on aggregate failure mechanisms. Since 1970, Petrography of Aggregates has been taught about every two to three years mostly for graduate students in civil engineering materials. Occasionally, engineering geology students also enroll in the course.

A general outline for the graduate level course is presented below in Table 4. References used in the course are Dolor-Mantuani, L. (1983), ASTM Annual Book of Standards, Concrete and Aggregates (1984); ASTM, STP169C, Concrete and Concrete Making Materials, 1994; Wahlstrom, 1951, Optical Crystallography, Kerr, 1959, Optical Mineralogy, and the Indiana Department of Transportation, Standard Specifications, 1993.

Table 4. Graduate Level Course, Purdue University
Petrography of Aggregates
by T.R. West

Lecture Subjects in Chronological Order

-Definitions, petrography and aggregate terminology
-Specific gravity and absorption of aggregates
-Sources and distribution of natural aggregates
-Field investigation of aggregate sources
-Standard laboratory highway testing of aggregates
-Deleterious materials in aggregates
-Aggregate petrography, megascopic
-Concrete failures, condition surveys
-Geology of pits and quarries
-Alkali silica reaction
-Alkali carbonate reaction
-Aggregate degradation
-Shape, roundness and surface properties
-Polishing of aggregates, bituminous pavements
-Soundness, freezing and thawing of aggregates and concrete
-Chemical resistance of concrete, sulfate attack
-Concrete petrography
-Air voids system in concrete
-Concrete and aggregate deterioration, a summary

Laboratory Subjects

-Mineral identification, common rock forming, economic, less common minerals
-Rock identification; igneous, sedimentary, metamorphic
-Field investigation, operating gravel pit
-Introduction to the petrographic microscope
-Opaque minerals, isotropic minerals
-Uniaxial minerals
-Biaxial minerals
-Thin sections of aggregates, igneous, sedimentary, metamorphic
-Petrography of concrete
-Field investigation, quarry or underground mine

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A Modified Presplit Blasting Method for use in Environmentally Sensitive Areas

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ABSTRACT

Engineering geologists of the Geotechnical Engineering Bureau of the New York State Department of Transportation (NYSDOT) are often called upon to design and assist in the construction of rock slopes along State highways. In the past, they had only two major concerns when designing a proposed rock cut, stability and cost. Recently, much emphasis has been placed upon a third consideration, aesthetics. Of particular concern are rock cuts located within the six million acre Adirondack State Park, wherein lies several hundred miles of State highway.

Activities within the Park are reviewed and administered by the Adirondack Park Agency (APA), including DOT projects, which are subject to APA approval. It is a goal of the APA to assure that any rock cuts constructed or modified within the Park blend to the greatest extent possible with the adjacent topography, and achieve or retain a "natural" look. The APA is especially opposed to the smooth, quarry-like appearance of constructed rockslopes such as those produced by presplitting, and the presence of conspicuous drill-butt traces. Their position is that such slopes do not look natural, and that the overall appearance of the Park suffers as a result.

In response to an APA request for rougher, more natural looking rock cuts, NYSDOT personnel produced a specification for a modified presplit blasting procedure. The result is a costlier method of blasting, which is essentially a compromise between constructing a least-cost stable rock face, and one that is aesthetically pleasing. It is our conclusion that the desired results can be achieved, but at higher costs, which are incurred by:

1) Increased presplit drilling
2) Increased excavation quantities
3) Increased post-blasting cleanup (scaling)
4) Possible increased maintenance

AREA GEOLOGY

The Adirondack Mountains form a roughly circular region in northeastern New York State. The Adirondack region is part of a much larger area, called the Grenville Province, which is an extensive belt of Precambrian basement rock, extending generally from Labrador to Mexico, and outcropping prominently in the Adirondacks. The rocks of the Adirondack region are almost exclusively metamorphic, with gneisses and anorthosites predominating. They have been severely folded and sheared, creating zones of extremely deformed mylonites. In addition, most rocks of the region display an abundance of joints. Typically, three sets of approximately mutually perpendicular joints are found, with spacing ranging from a few inches to several feet.

The deformation of these rocks occurred during the Grenville Orogeny, which took place approximately 1.1 billion years ago. During this mountain building event, the crust was severely deformed and thickened, and the rocks at depth were intensely metamorphosed. Over the next several hundred million years, erosion stripped away more than 15 miles of rock. The area was then covered by shallow seas, in which sediments accumulated and lithified through the Cambrian and Ordovician Periods. Sometime in the Tertiary, the Adirondack dome began to rise, and erosion then carved the area into the separate mountain ranges that we see today.

HISTORY OF THE PARK AND THE APA

On May 2, 1892, New York Governor Roswell P. Flower signed the Adirondack Park Enabling Act, which set aside 2.8 million acres to be "forever free" from the loggers' axe. Support for the Act was fueled not by sentiment for wildlife or aesthetics, but out of fear of a depleted water supply. The Adirondack Mountains contain the source waters for many of the State's principle waterways, and powerful interests felt that continued logging would degrade these avenues of commerce and travel.

In 1971, the New York State Legislature passed the Adirondack Park Agency Act, in order "to preserve the aesthetic and scenic value of the Park". By this time the Park had grown to its' present size of over 6 million acres, which encompasses several hundred miles of state highway. All activities within the Park are reviewed and administered by the APA, and are subject to APA approval. NYSDOT projects are no exception, and in the early 1990's several rock cuts were slated for remedial work. Bearing in mind their concern for aesthetics and scenery, and the DOT's need to construct safe, stable rock cuts, a compromise was made which would hopefully satisfy both parties.

MODIFIED (STAGGERED) PRESPLIT

THE SPECIFICATION

In early 1992, the head of the Engineering Geology section of NYSDOT met with representatives of the APA to discuss the options for constructing a rough, "natural" looking rock face for an upcoming project. It was agreed that in areas where rock was to be cut back exposing long, vertical faces, a staggered pattern of blast holes for presplitting would be specified, subject to review and adjustment. This procedure would give a rough appearance to the rock face while still achieving a linear break that could be counted on to provide a safe, stable rockslope. Also, it was hoped that if drill butt traces were visible in certain instances, they would blend in better due to the irregular texture of the rock.

NYSDOT has a standard specification for rock excavation, which, briefly stated, requires presplitting on any rockslope that is to be recut at an angle steeper than one vertical on one horizontal, and is greater than five feet in height. The specification calls for standard spacing of presplit holes, using a maximum column charge of 0.35 pounds per foot of presplitting explosive. The specification also requires the scaling of any fragmented or unstable rock from the blasted rock face.

By September of 1992, in response to the APA's request, a special specification had been produced by NYSDOT personnel, entitled "Modified Presplit", making certain changes in the standard specification. This specification reads in part:

> Two rows of presplit holes at the design angle will be drilled to toe of slope elevation plus subdrilling to contain the base charge. The front row of holes shall be located at the design finished slope offset.
>
> The front row holes shall be drilled on two and one-half foot centers. The back row holes shall be drilled two and one-half feet behind the front row on five foot centers, located behind alternate front row holes.
>
> <u>A base charge only shall be loaded into the front row holes located directly in front of the rear row holes.</u> The unloaded portion of all holes shall be completely stemmed with 1A (one quarter inch) crushed stone.
>
> Explosive initiation delays shall be timed to progress sequentially along the column loaded holes. The base charge only loaded holes in the front row shall be on the same delay as the next hole up section in the front row. No more than 25 milliseconds delay will be allowed between adjacent holes in the front row.
>
> The front row holes shall be the reference for the advancement of presplitting into the next production section.
>
> This modified presplit procedure shall be subject to adjustment, if deemed necessary, after the review of the test section by an Engineering Geologist.

Figure 1 is a plan view of a typical shot using the modified presplit method:

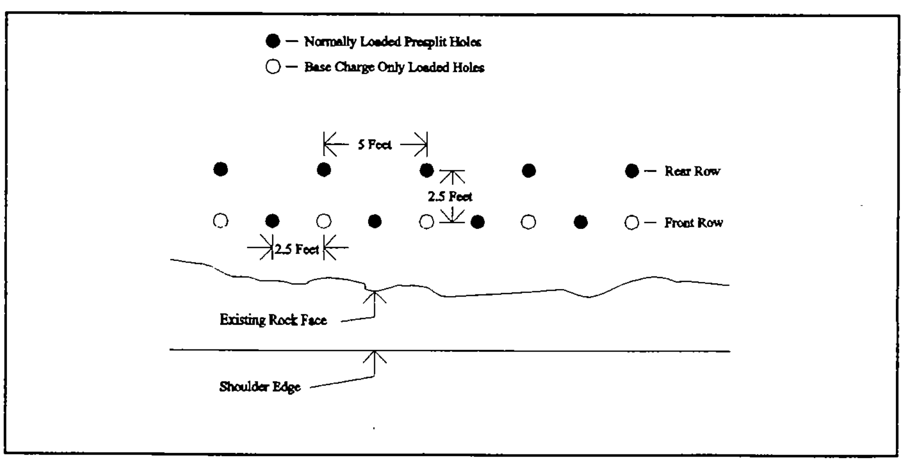

Figure 1

The location of the front row of holes is equivalent to that of the presplit line of a standard presplit slope, and is the reference for constructed final offset. If the section being shot is to be done in lifts, the base charge only holes are eliminated from the upper lifts, as are the base charges in the column loaded holes, in order to prevent damage to the final face midway up the slope. As with presplitting as normally done according to NYSDOT specifications, modified presplit holes are detonated prior to any production holes in that section.

THEORY

In theory, the pattern is designed to break the rock along the column loaded holes in a zig-zag fashion, with the spacing wide enough to prevent shearing along the back row of holes. If the rock were perfectly homogeneous and free of fractures, one would expect the end result to be a jagged, saw-toothed face, matching the layout of the presplit lines. However, the rock of this region is rarely homogeneous, and as previously noted, is heavily jointed. The detonation of two offset presplit lines is designed to interact with this jointing to produce a rough, yet stable, rockslope. The purpose of the base charge only loaded holes in the front row is to prevent high rock at the toe of the slope in the ditch.

PRACTICE

This procedure was used on three (3) NYSDOT contracts in 1994, all of which are located within the Adirondack State Park. The three projects are: (1) Route 10 in Caroga Lake, Fulton County, where blasting began in April, 1994, and ended in October, 1994 ; (2) Route 30 in Long Lake, Hamilton County, where blasting began in July, 1994, and is ongoing; and (3) Ticonderoga, Essex County, where blasting began in August, 1994, and was completed in October, 1994. On all three projects, test sections were shot according to specification. The note on adjustment of the procedure proved insightful, however, as problems were encountered almost immediately.

CAROGA LAKE PROJECT:

At Caroga lake, three slopes were cut using the modified presplit method, with a sum total of 1720 feet of presplit. 0.34 lb./ft. of presplit powder was used for the entire project. The entire first slope was used as a test section, subject to approval by a NYSDOT Engineering Geologist, and the APA. Upon inspecting the results of the initial blasts, it became apparent that the presplit line was shearing along the back row of holes. Instead of experimenting with changes, it was decided to complete this first slope according to the specification. Regardless of the problems encountered with the presplit blasting, a rough look was obtained, and this test section was accepted by both the APA and NYSDOT. Figure 2 is an overall view of this rockslope.

FIGURE 2

Hoping to prevent shearing along the back row of presplit holes, the next slope on this project was shot using a pattern of 6 foot spacing x 3 foot burden. Shearing proved minimal, and the results proved to be more in line with what the APA was looking for. Figure 3 shows a section of this rockslope, and illustrates how the use of staggered presplitting interacts with the jointing to produce a rough face. Based on these results, the 6 foot spacing x 3 foot burden was maintained for the remainder of the project.

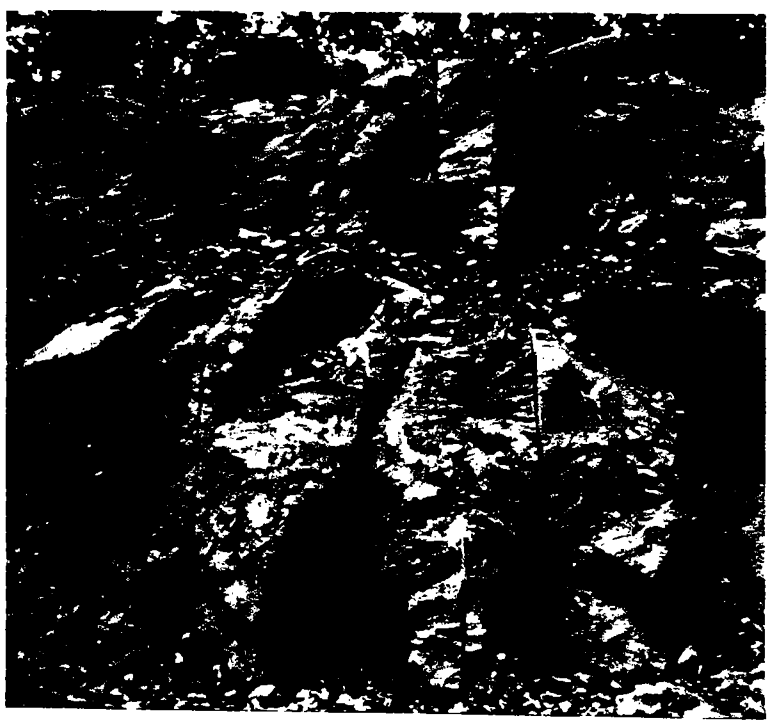

FIGURE 3

LONG LAKE PROJECT

At Long Lake, 24 rockslopes have been recut using the modified presplit technique, with a sum total of approximately 5750 feet of presplit. 0.25 lb./ft. of presplit powder was used for the entire project. Two slopes were cut using the specified 5 foot spacing x 2.5 foot burden, but again, shearing along the back row of holes was a problem (See Figure 4).

FIGURE 4

Based on the results obtained on these two slopes, the decision was made to widen the pattern to 6 foot spacing x 3 foot burden, and this pattern was maintained for the remainder of the project. In regard to the APA's wish to construct rough, "natural" looking rockslopes, we feel that many of the cuts on this project were a success. Figures 5 and 6 illustrate two such slopes.

FIGURE 5

FIGURE 6

Figures 7 and 8 show two slopes on this project which illustrate the effect of staggered presplitting and jointing combining to produce a rough, stable rockslope.

FIGURE 7

FIGURE 8

As previously noted, one hope of the APA was that this method of blasting would minimize the visual impact of the drill-butt traces. We feel that we were at least partially successful in this respect, as the following figures illustrate. Figures 9 and 10 show a typical cut at Long Lake. As seen by an approaching motorist, the drill-butt traces are barely visible (Figure 9) until directly upon the slope (Figure 10).

FIGURE 9

FIGURE 10

TICONDEROGA PROJECT

The project at Ticonderoga involved only one cut, 350 feet in length. Three test sections were shot on this slope, the first using the specified 5 foot spacing x 2.5 foot burden, and .33 lb./ft. of presplit powder. This test shot resulted in shearing along the back row of presplit holes. The pattern was widened to 6 foot spacing x 3 foot burden for the second test section, with no visible improvement in the results. In fact, this shot resulted in breakage behind the back row of presplit holes. Because of this, the presplit powder was changed from .33 lb/ft. to .25 lb/ft for the third test section, maintaining the 6 foot x 3 foot pattern. This change appears to have prevented any further breakage behind the presplit line, but shearing along the back row of holes continued. However, the test section was approved by a NYSDOT Engineering Geologist, and this blasting pattern and powder strength was used for the remainder of the project. Figure 11 shows the area of the second test section, illustrating the breakage behind the presplit line. Obviously, the nature of the jointing played a role in this breakage. Figure 12 shows how staggered presplitting and jointing can combine to produce the opposite of what is desired when the orientation of a major joint set closely matches the orientation of the finished presplit face.

FIGURE 11

FIGURE 12

CONCLUSIONS

It is our conclusion that this method of blasting can produce rough, stable rockslopes, but at added expense. The amount of presplit drilling required increases by approximately 50 percent, with corresponding increases in time and labor. The end results are unpredictable, varying between those that might be obtained by standard presplitting or by production blasting. Because rock is broken and removed from behind what would be the normal offset of a presplit plane, excavation quantities are increased and harder to estimate. Due to the fact that this method does not break the rock along a smooth payment line, increased post-blasting cleanup (scaling) is often required to stabilize the blasted rock face. And finally, because the end result is often a rough, uneven surface, there is potential for an increase in the rate of weathering. This may result in the rockslopes requiring maintenance sooner, and more often than had they been cut using standard presplit procedures.

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EFFECT OF SOIL HORIZONATION ON FLEXIBLE PAVEMENT RESPONSES

by

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and
Quintin B. Watkins

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ABSTRACT

Effect of Soil Horizonation on Flexible Pavement Responses

by

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Mechanistic pavement design refers to a process in which pavement *responses* (deflections, stresses, and strains) are used to determine adequate thicknesses of paving layers to carry projected traffic. In most mechanistic design schemes a structural pavement model is used to estimate pavement responses to traffic loads. Paving layer material properties and subgrade properties must be specified for the model. Typically the subgrade soil is modeled as a single layer with "fixed" properties (e.g. resilient modulus, Poison's ratio) for the entire layer thickness. However, many subgrade soils in fact exhibit distinct pedologic soil horizons. This paper investigates the effect on pavement responses of modeling the subgrade soil as a layered system, considering the soil's horizons.

Four soil series located in Champaign County, Illinois are used in the study. The resilient modulus for each horizon is estimated from soil "index" properties (% clay, plasticity index, % organic carbon), values for which are taken from the county soil survey published by the Soil Conservation Service. The ILLI-PAVE finite element pavement model is used to estimate three primary responses for conventional flexible pavements: (1) pavement surface deflections, (2) horizontal tensile strain at the bottom of the asphalt concrete surface, and (3) vertical compressive stress at the top of the subgrade soil. Horizontal tensile strain in the asphalt surface is used to estimate the fatigue life of the pavement. Vertical compressive stress in the subgrade is used to estimate rutting susceptibility of the pavement structure.

The results indicate the most consistent method of considering soil properties in mechanistic models is to include properties from both the B and C horizons of the soil. However, fatigue life and rutting susceptibility estimates obtained when using soil property based estimates of subgrade resilient modulus are unconservative relative to estimates obtained when using FWD deflection based subgrade modulus estimates.

Effect of Soil Horizonation on Flexible Pavement Responses

by

Kevin D. Hall and Quintin B. Watkins

INTRODUCTION

Subgrade soils play a crucial role in the design and performance of flexible pavements. Most pavement design procedures include a "subgrade input", whether as an explicit variable or implicitly in design curves. In most design situations, the subgrade soil is treated as a single material, with one set of values to represent pertinent soil properties. This "global" approach to considering subgrade properties is perfectly reasonable for higher-volume roads, in which the pavement structure is sufficiently stiff to limit the effects of the subgrade, and in cut/fill situations, in which the subgrade soil is basically "constructed" beneath the pavement.

Characterization of subgrade soils becomes especially critical, however, in the context of low-volume roads, particularly those constructed directly over the "natural" subgrade. In these situations subgrade conditions often govern the performance of the pavement. The use of a "global" approach to characterizing the subgrade soil as a single material with a single set of properties may not be appropriate. Soils typically exhibit distinct pedologic *horizons* in their natural state. The engineering properties of horizons for some soils can be quite dissimilar.

This paper investigates the effect on flexible pavement responses to load of characterizing subgrade soils as layered materials with distinct layer properties, versus characterizing soils as single layers with global properties. The results may help shed light on approaches to considering the subgrade soil in design of flexible pavements.

MECHANISTIC PAVEMENT DESIGN

Prior to looking at the effect of subgrade soil horizons, it is helpful to review some concepts pertaining to mechanistic pavement design. *Mechanistic* pavement design refers to a process in which pavement *responses* (surface deflections, stresses, and strains) are used to determine adequate thicknesses of paving layers to carry projected traffic. Figure 1 is a flow diagram outlining the typical steps in a mechanistic design process *(NCHRP 1-26, 1990)*. The two primary components of the process are the STRUCTURAL MODEL and TRANSFER FUNCTIONS. The

STRUCTURAL MODEL is an analysis procedure that uses INPUTS (materials, soils, climate, traffic, etc.) to predict PAVEMENT RESPONSES (stresses {σ}, strains {ε}, deflections {δ}) in the pavement system. Examples of STRUCTURAL MODELs include elastic layer procedures (BISAR, ELSYM5, etc.) and finite-element procedures (ILLIPAVE, etc.). TRANSFER FUNCTIONS relate PAVEMENT RESPONSES to the number of load applications (N) a pavement can carry to some state of "failure" as determined by pavement distress such as fatigue cracking or rutting *(Elliott, et al, 1990).*

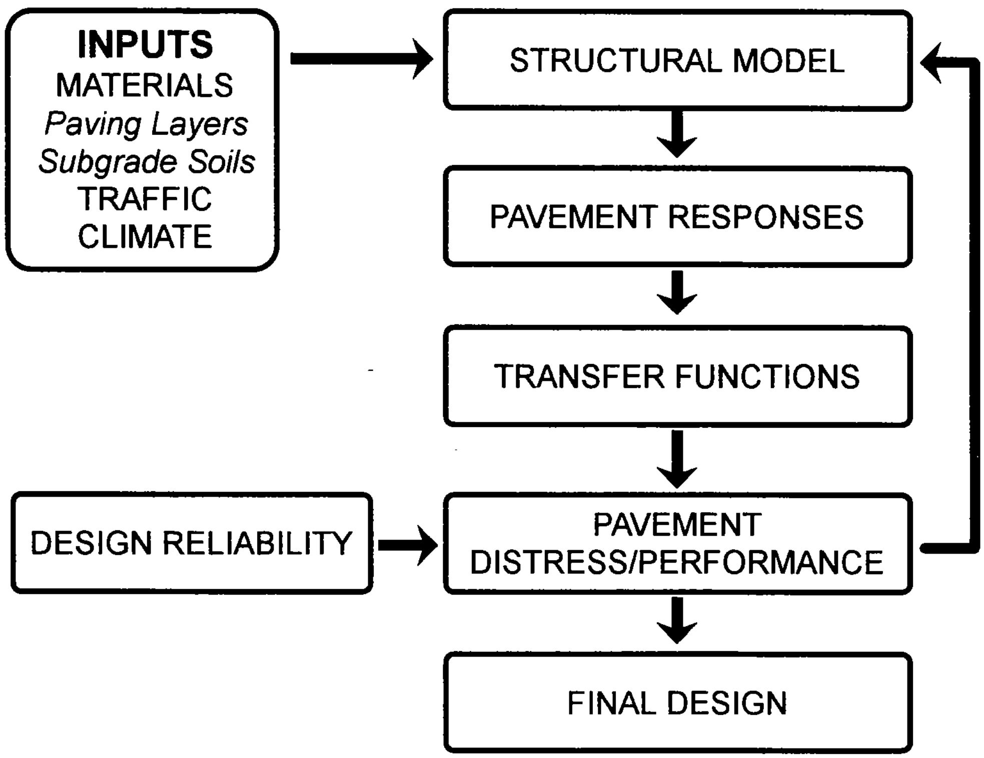

Figure 1. Mechanistic Design Flowchart

Mechanistic Design for Flexible Pavements

The two primary load-related distresses in flexible pavements are fatigue cracking in the asphalt concrete (AC) surface and rutting. AC fatigue cracking is generally related to the maximum tensile strain in the AC layer. Rutting is the accumulation of permanent strain in each of the paving layers and the subgrade. A "check" procedure in the design process is frequently used to evaluate the low-temperature cracking potential of the AC, which is not load-related.

Asphalt Concrete Fatigue. Tensile strain in the AC surface is affected by the level of "support" offered by the subgrade soil, as measured by the subgrade resilient modulus (Hall, 1993). The TRANSFER FUNCTION for asphalt concrete fatigue, which relates AC tensile strain to the allowable number of load applications to fatigue failure, is critical to the mechanistic design procedure *(NCHRP 1-26, 1992)*. There is not a "unique" fatigue algorithm suitable for all AC mixtures. Typical forms for AC fatigue algorithms are shown as Equation 1 (strain based) and Equation 2 (strain/modulus based).

$$N = k_1 * (1/\varepsilon)^{k2} \quad \textbf{(1)}$$

$$N = k_1 * (1/\varepsilon)^{k2} * (1/E)^m \quad \textbf{(2)}$$

where: k_1, k2, m = experimentally determined parameters
ε = AC tensile strain
E = AC resilient modulus

The AC fatigue algorithm used in the Illinois DOT full-depth asphalt mechanistic design procedure is shown as Equation 3 *(Illinois DOT, 1989)*. This algorithm is used in this research to estimate the number of load applications to fatigue failure resulting from applied AC tensile strain.

$$N = 5x10^{-6} * (1/\varepsilon_{AC})^{3.0} \quad \textbf{(3)}$$

where: ε_{AC} = maximum tensile strain in AC surface (in/in)

Rutting. A rutting TRANSFER FUNCTION used successfully in Illinois and considered in NCHRP 1-26 *(1990)* is the *subgrade stress ratio* (SSR). The subgrade stress ratio is defined as the ratio of the deviator stress applied to the subgrade, to the unconfined compressive strength of the subgrade soil. Equation 4 shows an algorithm developed for flexible pavements to predict SSR from pavement surface deflections *(Elliott and Thompson, 1985)*. This algorithm is used in this research to assess the rutting potential of pavements.

$$\text{Log SSR} = 1.671 * \log \delta - 2.876 \quad \textbf{(4)}$$

where: δ = maximum surface deflection, mils

The SSR can be used as an indicator of rutting potential *(NCHRP 1-26, 1992)*. The typical application of SSR is the use of a "threshold" SSR value, above which rutting is most likely to occur. Threshold values for SSR are established by a pavement agency, and can range from 0.4 to 0.7, depending on the "criticality" of the rutting estimate *(University of Illinois, 1992)*.

Subgrade Soils Models. The properties of subgrade soils are considered as INPUTS to the STRUCTURAL MODEL in a mechanistic design procedure. Typically, the properties used include the resilient modulus (E_R) and Poisson's ratio (μ). The form of the particular pavement and materials model(s) govern the properties required.

Fine-grained soils typically exhibit "stress-softening" behavior under repeated loads. That is, the resilient modulus decreases with increasing applied deviator stress. This behavior has been modeled as a bi-linear curve, as shown in Figure 2 *(Thompson and Robnett, 1979)*. For Illinois fine-grained soils the "breakpoint" of the curve occurs consistently around 6 psi deviator stress. The resilient modulus corresponding to this point is referred to as the "breakpoint" resilient modulus, E_{ri}. E_{ri} is a good indicator of the soil's resilient behavior and is the dominant influence on pavement structural responses *(NCHRP 1-26, 1992)*.

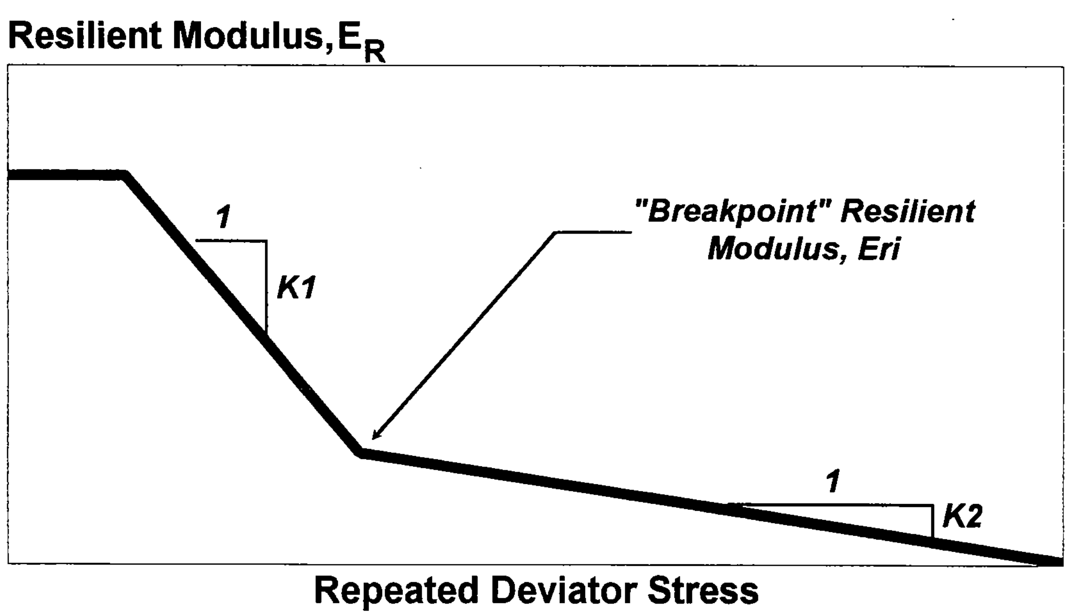

Figure 2. Subgrade Resilient Modulus Bi-Linear Model.

RESEARCH METHODOLOGY

The research presented here is comprised of three primary tasks: (1) determination of soil horizons and properties for typical fine-grained subgrade soils; (2) estimation of flexible pavement responses, using soil horizons and properties identified in task #1; (3) comparison of pavement responses generated using soil horizon combinations with responses generated using data obtained with a falling-weight deflectometer (FWD). Detailed descriptions of each of the tasks follow.

Determination of Soil Horizons and Properties

An extensive non-destructive pavement testing program was conducted in 1992 on county roads in Champaign County, Illinois as part of a project to characterize Illinois subgrade soils *(Hall, 1993)*. The four pedologic soil series comprising the largest area of Champaign County are selected from that program to demonstrate horizon effects: 56 Dana, 146 Elliott, 152 Drummer, and 154 Flanagan. The *Soil Survey of Champaign County Illinois* (1990) is used to determine the arrangement and extent of genetic horizons in each of the four soil series. Table 1 shows the number, arrangement, and thickness of horizons for the series. The Champaign County soil survey also lists estimated ranges of soil properties for each soil series encountered in the county, as well as information relating to classification. Table 1 shows range "midpoint" values for plasticity index (PI), clay content (%CL), and organic carbon content (%OC), and the U.S. Department of Agriculture (USDA) textural classification for the four subject soils in this project.

Soil Series	Horizons		USDA Textural Class	Average Values			Eri <OPT> (ksi)	Eri <Design (ksi)
	Thick (in)	Sym		PI (%)	%CL	%OC		
56 Dana	12	A	SiL	10	17	1.50	6.2	1.7
	22	B	SiCL	26	31		10.6	8.5
	5	IIB	CL	24	31		10.3	7.3
	21	IIB/IIC	L	8	21		7.5	1.0
146 Elliott	12	A	SiL	13	26	2.65	4.7	1.0
	29	IIB	SiC,SiCL	19	40		10.6	7.1
	19	IIC	SiCL,CL	18	31		9.6	6.1
152 Drummer	14	A	SiCL	23	31	3.50	5.1	1.6
	27	B	SiCL	23	31		10.2	6.7
	6	IIB	L,SiL,CL	23	28		9.8	1.0
	13	IIC	SaL/SiCL	14	24		8.4	4.9
154 Flanagan	18	A	SiL	23	25	2.65	6.7	1.0
	27	B	SiCL	23	39		10.9	7.4
	15	IIB/IIC	L,CL,SiL	18	25		9.0	1.0

Table 1. Soil Properties for Champaign County Soils

A number of studies have been conducted to develop subgrade resilient modulus predictive equations based on soil "index" properties such as Atterberg limits, percent clay, etc., and the stress-state of the soil *(Thompson & Robnett, 1979; Drumm, et al, 1990; Elliott, et al, 1988; Farrar & Turner, 1991)*. For this research, modulus predictive equations developed for Illinois fine-grained soils by Thompson and Robnett *(1979)* are used to estimate the resilient modulus for each horizon of the four subject soils. These equations, shown as Equations 5 and 6, have been used in other studies of Illinois soils *(Thompson & LaGrow, 1988; Hall, 1993)*, and recognized as viable methods of estimating subgrade resilient modulus *(NCHRP 1-26, 1990)*.

$$E_{Ri(OPT)} = 4.46 - 0.098 * \%CL + 0.119 * PI \qquad \textbf{(5)}$$

$$E_{Ri(OPT)} = 6.9 + 0.0064 * \%CL + 0.216 * PI - 1.97 * \%OC \qquad \textbf{(6)}$$

where: $E_{Ri(OPT)}$ = "breakpoint" subgrade resilient modulus (ksi) at optimum moisture content and 95% compaction
%CL = percent clay (< 0.002 mm)
PI = plasticity index (%)
%OC = percent organic carbon

Table 1 shows estimates of $E_{Ri(OPT)}$ for each of the horizons of each of the subject soils. Equation 6 is used for A-horizons (which contains a relatively significant amount of organic carbon), while Equation 5 is used for all other horizons (which typically do not contain significant amounts of organic carbon).

Equations 5 and 6 estimate the resilient modulus of subgrade soils at optimum moisture content. For fine-grained soils, moisture content significantly affects the resilient modulus *(Thompson & Robnett, 1979)*. *In-situ* subgrade soils typically exhibit moisture contents well above optimum; an adjustment of the E_{ri} estimate is necessary to account for field moisture conditions *(Thompson & LaGrow, 1988; Hall, 1993)*. Thompson and LaGrow *(1988)* developed "moisture adjustment factors" for fine-grained Illinois soils based on the soil's USDA textural classification. The adjustment factors are as follows: USDA "silt" - 2.1; USDA "silt loam" - 1.5; USDA "clay", "silty clay" and "silty clay loam" - 0.7. The moisture adjustment factor is subtracted from the $E_{Ri(OPT)}$ estimate for each moisture percentage point above optimum. Based on analyses by Hall *(1993)*, a typical moisture value for 56 Dana was assumed to be "OPT+3" (a moisture content 3% above optimum moisture), while an "OPT+5" condition was assumed for

146 Elliott, 152 Drummer, and 154 Flanagan. Table 1 shows the resilient modulus estimate for each horizon of each soil based on the soil's "design" moisture content.

Estimation of Flexible Pavement Responses

As previously discussed, a number of STRUCTURAL MODELS are available to estimate stresses, strains, and deflections for a given pavement structure. For this research the ILLIPAVE structural pavement model is used. ILLIPAVE is a finite-element method that incorporates stress-dependent material models. The choice of STRUCTURAL MODEL is not critical *(NCHRP 1-26, 1992)*; the ILLIPAVE model has been used for a number of pavement studies in Illinois and elsewhere *(i.e. Elliott & Thompson, 1985, Gomez-Achecar & Thompson, 1986)*.

The focus of this research is the effect of the subgrade on pavement responses. Accordingly, a "generic" pavement structure is used for all analyses, consisting of 3-inches asphalt concrete surface and 8-inches granular (crushed stone) base course. Material properties for the asphalt concrete and crushed stone are identical to those used by Elliott and Thompson *(1988)* for conventional flexible pavements. Load applied to the pavement in the model is similar to that applied by a falling weight deflectometer (9000 lb applied across a circular radius of 5.9055 inches). This loading approximates a 9000-lb wheel load on the pavement.

Pavement responses recorded from ILLIPAVE analyses include the maximum tensile strain in the asphalt concrete surface, the maximum vertical compressive stress applied to the subgrade, and pavement surface deflections directly beneath the load and at distances from the load of 12 inches, 24 inches, and 36 inches. Equation 3 is used with the maximum tensile strain to estimate the fatigue life of the pavement. Equation 4 is used with the maximum deflection under load to estimate the subgrade stress ratio (SSR) of the pavement.

The subgrade soil was modeled in ILLIPAVE as a stress-dependent material with the characteristics shown in Figure 2. Values for the respective slopes for the straight-line portions of the soil model are reproduced from Elliott and Thompson *(1985)*. These slopes, which define the degree of stress-dependency of the soil, are held constant for all horizons and all soils. The breakpoint resilient modulus used in ILLIPAVE is taken from the "design" values listed in Table 1. The total subgrade thickness (depth) is 120 inches. For multi-layered subgrades the C-horizon for all soils is extended to produce the total 120-inch subgrade thickness.

Comparison with FWD-based Data

Deflection data measured using an FWD in Champaign County are used to "backcalculate" or estimate the resilient modulus of subgrade soils *in-situ*. This resilient modulus estimate reflects the actual moisture and stress-state conditions experienced by the soil in the field at the time of testing. Hall *(1993)* gives a complete description of the FWD testing program and resulting modulus estimates for Champaign County soils. Additional ILLIPAVE analyses are performed using the FWD-based estimates of resilient modulus. These analyses should provide the best available estimate of the behavior of pavements in the field. This is not to imply that FWD-based modulus estimates are "correct"; it is however a recognition that deflection-based modulus estimates are the "best available" estimates of *in-situ* subgrade modulus.

Pavement responses resulting from the use of soil property based estimates of subgrade modulus are compared to responses resulting from the use of deflection-based modulus estimates. A variety of horizon combinations are used in the comparisons, including "all horizons", "B and C horizons only", "B horizon only", and "C horizon only". Such comparisons serve two purposes. The first is to quantify the effect of horizonation on pavement responses. The second is to identify the single horizon (or combination of horizons) whose properties (and hence, modulus estimate) result in pavement responses that best approximate the pavement responses resulting from the use of deflection-based modulus estimates.

RESULTS

Table 2 summarizes the results of the ILLIPAVE analyses, including the maximum surface deflection, AC strain, applied subgrade stress, estimated fatigue life, and subgrade stress ratio. Table 3 shows the results of comparisons between pavement responses generated using soil property based modulus estimates and responses generated using FWD deflection based estimates. The ratios shown in Table 3 are relative to the FWD based results; for example "B/FWD" indicates the numbers shown are the ratio of results from the "B horizon only" analysis to the FWD analysis.

Figures 3 and 4 graphically represent the ratios shown in Table 3 for fatigue life and SSR. The trends exhibited in the data presented in Figures 3 and 4 are typical for other data sets such as AC strain, applied subgrade stress, etc.

Soil Series	Horizon Combination	Maximum Deflection (mils)	AC Strain ($\mu\epsilon$)	Subgrade Stress (psi)	Fatigue Life (# apps)	SSR
56	All Horiz	30.3	372	5.13	97,127	0.398
Dana	B&C Only	26.4	342	10.10	124,995	0.316
	B Only	23.4	337	10.70	130,641	0.258
	C Only	35.4	385	5.48	87,617	0.516
	FWD	28.8	361	8.26	106,279	0.365
146	All Horiz	29.6	377	5.48	93,314	0.383
Elliott	B&C Only	24.7	343	10.10	123,905	0.283
	B Only	24.7	344	10.10	122,827	0.282
	C Only	25.7	348	9.60	118,640	0.303
	FWD	25.1	345	9.91	121,762	0.290
152	All Horiz	29.5	372	6.67	97,127	0.380
Drummer	B&C Only	25.8	349	9.55	117,623	0.304
	B Only	25.1	345	9.91	121,762	0.290
	C Only	27.3	355	8.89	111,760	0.333
	FWD	30.4	367	7.63	101,151	0.401
154	All Horiz	32.1	380	5.81	91,121	0.438
Flanagan	B&C Only	27.3	347	9.66	119,669	0.333
	B Only	24.4	342	10.20	124,995	0.276
	C Only	35.4	385	5.48	87,617	0.516
	FWD	28.2	358	8.51	108,974	0.352

Table 2. Results of ILLIPAVE Analyses

Soil Series	Horizon Combination	Maximum Deflection (mils)	AC Strain ($\mu\epsilon$)	Subgrade Stress (psi)	Fatigue Life (# apps)	SSR
56	All/FWD	1.05	1.03	0.62	0.91	1.09
Dana	B&C/FWD	0.92	0.95	1.22	1.18	0.86
	B/FWD	0.81	0.93	1.30	1.23	0.71
	C/FWD	1.23	1.07	0.66	0.82	1.41
146	All/FWD	1.18	1.09	0.55	0.77	1.32
Elliott	B&C/FWD	0.99	0.99	1.02	1.02	0.98
	B/FWD	0.98	1.00	1.02	1.01	0.97
	C/FWD	1.03	1.01	0.97	0.97	1.05
152	All/FWD	0.97	1.01	0.87	0.96	0.95
Drummer	B&C/FWD	0.85	0.95	1.25	1.16	0.76
	B/FWD	0.82	0.94	1.30	1.20	0.72
	C/FWD	0.90	0.97	1.17	1.10	0.83
154	All/FWD	1.14	1.06	0.68	0.84	1.25
Flanagan	B&C/FWD	0.97	0.97	1.14	1.10	0.95
	B/FWD	0.87	0.96	1.20	1.15	0.78
	C/FWD	1.26	1.08	0.64	0.80	1.47

Table 3. Results of Soil Property Based and FWD Based Response Comparisons

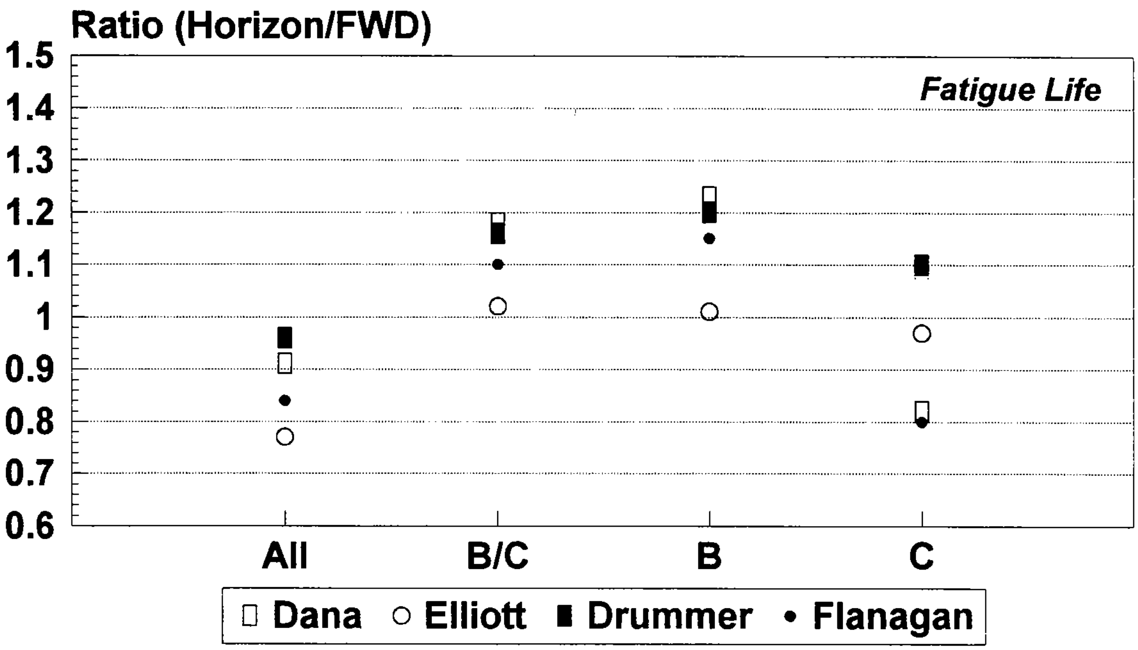

Figure 3. Comparisons of Soil Property Based and FWD Based Responses - Fatigue Life

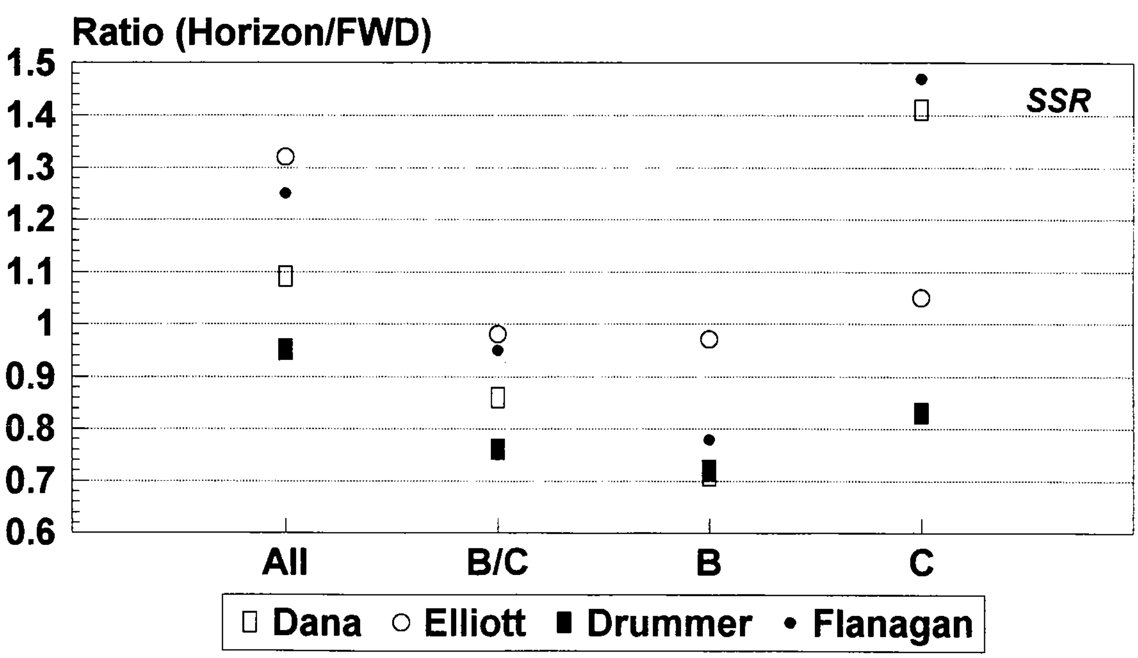

Figure 4. Comparisons of Soil Property Based and FWD Based Responses - SSR

CONCLUSIONS

The primary pavement responses of interest are the AC tensile strain and the load-induced deviator stress applied to the subgrade. The significance of these responses is the effect on response-related pavement distress potential and performance life. Figures 3 and 4 show the effects of modeling subgrade soils with horizons on the fatigue life and subgrade stress ratio of the pavement system, relative to modeling subgrade soils with the "best available" estimate of subgrade modulus -- backcalculated from pavement deflections measured in the field. Based on the data shown in Figures 3 and 4, and Tables 2 and 3, the following conclusions are offered:

- The most consistent grouping of fatigue life ratios for the four soils investigated occurs when subgrade soils are modeled using the properties from the B/C horizons. This indicates that this modeling technique is less sensitive to the particular soil series under consideration. The least consistent grouping occurs when using the C-horizon properties only, indicating a higher sensitivity to the particular soil series being considered.
- The fatigue life estimate obtained when modeling soils using the B/C horizons and the B horizon only is consistently higher than when modeling the subgrade using deflection-based modulus estimates. The fatigue life obtained when modeling soils using All horizons is consistently lower than with the deflection-based estimates.
- The B/C horizon based SSR estimate is more consistent across the range of soils investigated than other horizon groupings, indicating less sensitivity to the particular soil series used in the analysis. The C horizon based SSR estimates show the least consistency across the range of soil series.
- The B/C horizon and B-horizon based SSR estimates are consistently lower than the SSR estimates resulting from deflection-based subgrade modulus estimates. It should be noted, however, that few of the SSR estimates obtained might be considered "critical" (SSR>0.4) with respect to rutting potential.

RECOMMENDATIONS

For many "higher-type" pavements the designer is afforded the luxury of having relatively reliable estimates of *in-situ* soil conditions in the form of FWD based pavement surface deflections. Unfortunately, this is not always the case, and particularly so for lower-volume roads. Soil property information in these cases must be obtained elsewhere. When a designer is faced with a variety of soil properties and horizons, such as is shown in Table 1, the inevitable question arises: "which set of soil properties is suitable for use to estimate pavement responses?". **Based on the analyses and conclusions presented here, the most consistent method of modeling soils is to include properties from both the B and C horizons.**

The recommendation listed in the previous paragraph is not surprising. Most A-horizon soils tend to be highly organic, often highly compressible, and exhibit the largest variation in engineering properties. For these reasons, the A horizon "topsoil" is typically removed prior to roadway construction. It's inclusion in a pavement model is generally not advisable. Using the properties from only the B horizon or only the C horizon in a pavement model discounts the potentially very real differences in properties between the horizons, which is suggested by the data shown in this study. It should be noted that the soils used in this study may be considered to be fairly "consistent" in their soil profile relative to other soil series; that is, many soil series will display a larger variety in properties among their horizons that those used in this study.

The designer should be aware of the fact that the use of soil property based modulus estimates, even when coupled with the use of distinct horizon properties, do not provide the same degree of "reliability" that is provided by deflection-based modulus estimates. As shown here, the fatigue life estimated using soil-property based moduli is overestimated relative to deflection-based moduli, while soil-property related SSR is underestimated relative to deflection-related SSR values. Both of these trends would be considered "unconservative" in a design sense, and should be recognized and accounted for in design.

The conclusions and recommendations offered must be tempered by some cautions related to the scope of the study, as follows:

- Only four soils are used in the study, all existing within a single county in central Illinois. Soil series from other locations may exhibit different trends.
- A single level of stress-dependency was used in all soil modeling. Variations in the stress-dependency of soils, while not a major factor in pavement responses, will nonetheless affect the results.
- Soil property based resilient modulus estimates used in this study are based on assumed moisture content values that appear reasonable for the associated FWD testing conditions. Moisture content greatly affects subgrade modulus; changes in moisture content may affect the results of the analyses shown.

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PREDICTING THE UNCONFINED COMPRESSIVE STRENGTH OF CARBONATE ROCKS FROM LOS ANGELES ABRASION TEST DATA

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Abstract

Dry density, absorption, unconfined compressive strength, and Los Angeles abrasion tests were performed on fifteen different carbonate rocks that ranged in age from Pennsylvanian to Cambrian and in strength from relatively weak limestone to very strong micrite. Regression analyses were performed to determine the relationships that dry density, absorption, and Los Angeles abrasion loss may have with unconfined compressive strength. Results indicate that multiple linear regression, with unconfined compressive strength as the dependent variable and Los Angeles abrasion loss, dry density, and absorption as the independent variables, yields a useful predictive equation (adjusted $R^2=0.729$) for the rocks studied. The equation was found to be a valid predictor of compressive strength when data from five additional samples were used as input.

INTRODUCTION

The unconfined compression test and the Los Angeles (L.A.) abrasion test are the two most commonly performed tests in highway engineering. The tests are used to evaluate the quality of crushed stone for such applications as concrete aggregates, basecourse material, and railroad ballast. While it may be assumed that a rock of high unconfined compressive strength would suffer a relatively low amount of degradation in the L.A. abrasion test, there is a need to quantify the relationship between the results of the two tests, especially for carbonate rocks which serve as the most common source of concrete aggregates. The establishment of an empirical relationship between the results of the two tests would be of great interest because both tests require meticulous, time consuming, sample preparation and costly equipment. Time and money may be saved by obtaining reliable estimates of performance in one test from results of the other test.

Only a limited number of studies have been conducted to investigate the relationship between unconfined compressive strength and L.A. abrasion loss. Kazi and Al-Mansour (1980) found a relatively high degree of correlation between mean values of unconfined compressive strength (determined by a Schmidt hammer) and L.A. abrasion loss for igneous and metamorphic rocks. However, they did not develop an equation for the relationship. Cargill and Shakoor (1992) found a log-log relationship ($r=-0.92$) between unconfined compressive strength and L.A. abrasion loss (normalized with respect to dry density) for a small number of different rock types. The only study devoted entirely to carbonate rocks was conducted by Ballivy and Dayre (1984) who found an inverse, non-linear, relationship between unconfined compressive strength and L.A. abrasion loss for high-porosity ($n > 10\%$) carbonate rocks. However, neither a predictive equation nor any tests of significance were reported by these investigators.

The main objective of the present study was to determine if a simple, reliable equation for predicting unconfined compressive strength from L.A. abrasion loss, dry density, and percent absorption could be developed for carbonate rocks.

METHODOLOGY

Fifteen samples, each consisting of two or three large-size blocks of carbonate rocks of varying lithologies and ages were collected from sites in Ohio, Indiana, Kentucky, Pennsylvania, and Virginia (Figure 1). Care was taken to ensure that all blocks of a given rock were retrieved from the same stratigraphic unit. The blocks were cored in the

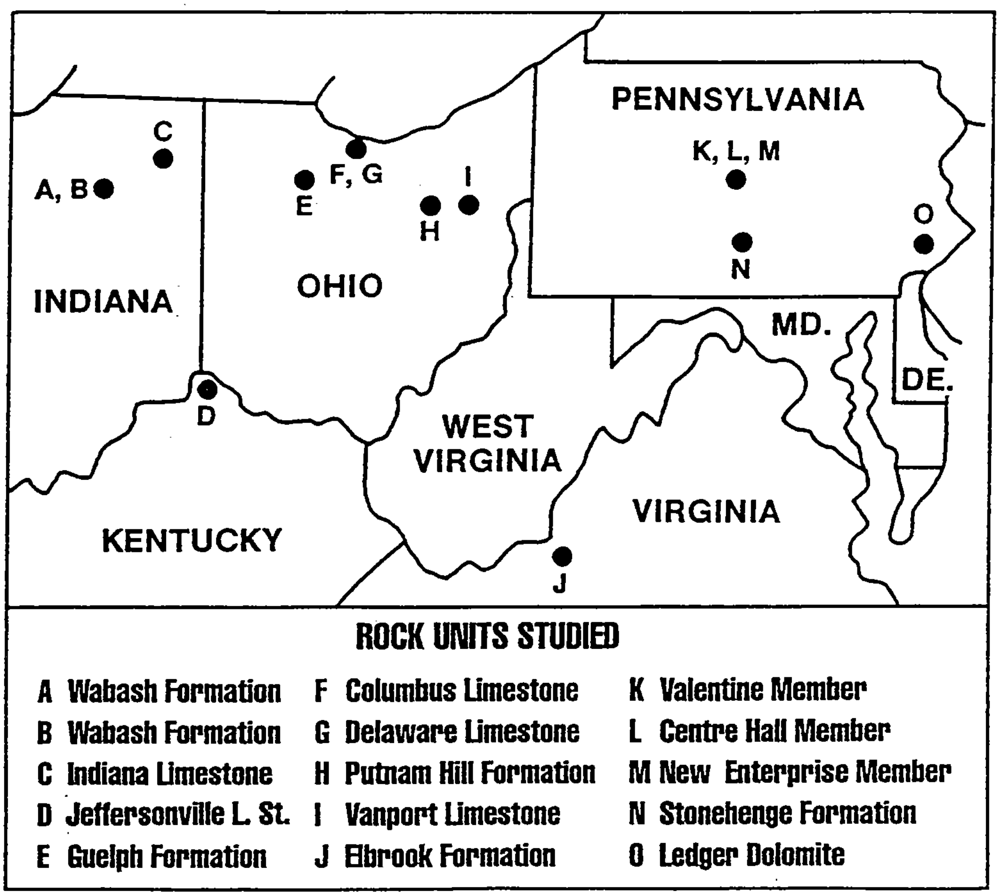


Figure 1. Locations of sampling sites.

laboratory to obtain approximately ten NX-size (54 mm) cores of each rock. Each core was trimmed and lapped to meet the specifications set by the American Society for Testing and Materials (ASTM) method D4553 (ASTM, 1990). The prepared cores had length to diameter (L/D) ratios ranging between 2.0 and 2.5. All cores were oven dried at 105°C for 24 hours and cooled to room temperature before testing. The aggregate for abrasion testing was obtained by breaking what remained of the rock blocks after coring. The aggregate was also oven dried at 105°C for 24 hours and cooled to room temperature.

Dry density and percent absorption were determined for each core prior to compression testing. Dry density was obtained by dividing the weight of each core by its volume whereas ASTM method C97 (ASTM, 1990) was used for determining absorption. Upon completion of the absorption test, the cores were again oven dried at 105°C for 24 hours and then subjected to unconfined compression test following ASTM procedure D2938 (ASTM, 1990). As recommended by Yamaguchi (1960), the unconfined compressive strength was measured on ten, or close to ten, cores of each rock type and the average values were obtained. ASTM method C131 (ASTM, 1988) was used to determine the L.A. abrasion loss on aggregate samples prepared to meet grading specifications (size range 9.5 mm to 25.0 mm). Three tests for each rock sample were performed and average values of L.A. abrasion loss were obtained.

A detailed statistical analysis was performed on test data using MBDP (1988) statistical software. The purpose of statistical analysis was to determine the summary statistics (mean, standard deviation, coefficient of variation, standardized skewness, and kurtosis) for each property and to investigate the nature of relationship that unconfined compressive strength may have with dry density, percent absorption, and L.A. abrasion loss. In order to discern patterns in test data, bivariate plots of unconfined compressive strength and L.A. abrasion loss as functions of dry density and percent absorption, and a plot of unconfined compressive strength as a function of L.A. abrasion loss, were generated. These plots did not suggest any model other than linear regression to explain the bivariate regressions. The summary statistics and bivariate plots indicated that the highest degree of uncertainty existed in the unconfined compressive strength data and, therefore, unconfined compressive strength was chosen as the dependent variable.

Based on a review of summary statistics and bivariate plots, an "all possible subsets" linear regression model (BMDP 9R; BMDP, 1988) was chosen to find the best predictor of unconfined compressive strength as a function of dry density, percent absorption, and L.A. abrasion loss. The strength of each regression was evaluated on the basis of its adjusted R^2 values. Once the best regression was obtained, its statistical significance at 95% confidence level was assessed by performing 't' and 'F' tests. A relationship was considered not to be significant if it failed the 'F' test or at least one 't' test. Any significant relationship was subjected to an analysis of residuals to confirm if the chosen model is appropriate. Residuals are deviations of observed values from predicted values and represent the errors in prediction.

RESULTS AND DISCUSSION

Regression analyses and tests of statistical significance were performed on the mean values of the four properties tested (dry density, percent absorption, L.A. abrasion loss, and unconfined compressive strength) as well as on the entire data. The mean values of the four properties are listed in Table 1. Bivariate Regression analysis did not result in statistically significant relationships between any of the properties measured. However, application of "all possible subset" regression model to mean values data yielded a relatively strong correlation (adjusted $R^2=0.729$) between unconfined compressive strength as the dependent variable and L.A. abrasion loss, dry density, and percent absorption as the independent variables. The results of multivariate analyses are shown in Table 2. The application of multivariate regression analysis to all data weakened the relationship, with adjusted R^2 value dropping to 0.461. Figures 2 and 3 show the results of residual analysis for the regression reported in Table 2. The distribution of data points in Figure 2 has the desired scattershot appearance. Additionally, although the data points in Figure 3 are not in a line along the axis, they do not exhibit an obvious pattern that would negate the assumption of normal distribution of error which is a requirement for this type of analysis.

The predictive equation using the multivariate regression model can be expressed as follows:

$$q_u=-5.8(\text{L.A. loss})+440.6(\rho)+32.5(\text{Abs.})-895.8$$

where: q_u = unconfined compressive strength in MPa
L.A. loss = Los Angeles abrasion loss in percent
ρ = density in g/cm^3
Abs. = absorption in percent

Since over 70 percent variation in the data is explained by the regression model (adjusted $R^2=0.729$), the equation given above may be considered as a useful predictor of unconfined compressive strength. The ranges of values of various properties (Table 2) for the rocks tested in this study are too narrow (L.A. abrasion loss = 18.6-37.2%; dry density = 2.43-2.83g/cm^3; absorption = 0.08-4.33%) to yield a higher degree of correlation. Previous studies (Smorodinov et al., 1970; Hugman and Friedman, 1979; Ballivy and Dayre, 1984; Cargill and Shakoor, 1991) show that weaker correlations existed whenever the range of independent variable was narrow. Considering the relatively narrow range of independent variables in the present study, the results of the present study are quite comparable with those of the previous studies for similar ranges of data. It should be noted that although increasing the range of independent variables, such as L.A. abrasion loss approaching 100 percent in Ballivy and Dayre's study (Ballivy and Dayre, 1984) would improve the correlation coefficient (R^2) value, the wider range will not be representative of the rocks commonly used in engineering practice, especially highway engineering. For example, specifications by ASTM (ASTM, 1990) require that L.A. loss should not exceed 40 percent for concrete aggregates (as is true of rocks tested in this study). Therefore, there will be less interest in predicting compressive strength for rocks with high L.A. abrasion loss values.

Table 1: Mean values of each property measured on Samples A through O.

Sample	Formation Name	Dry Density (g/cm^3)	Absorption (%)	Unconfined Compressive Strength (MPa)	L.A. Abrasion Loss (%)
A	Wabash Fm.	2.55	2.49	110.6	30.2
B	Wabash Fm.	2.43	4.33	134.8	32.2
C	Indiana Limestone	2.51	1.58	34.2	33.9
D	Jeffersonville Limestone	2.70	0.24	112.8	30.2
E	Guelph Fm.	2.49	1.90	69.1	37.2
F	Columbus Limestone	2.68	0.38	185.9	21.0
G	Delaware Limestone	2.64	0.40	137.7	23.9
H	Putnam Hill Fm.	2.68	0.32	159.2	22.8
I	Vanport Limestone	2.64	0.51	222.6	18.6
J	Elbrook Fm.	2.82	0.22	176.0	27.4
K	Valentine Member	2.70	0.13	112.9	35.3
L	Centre Hall Member	2.70	0.09	133.5	24.7
M	New Enterprise Member	2.68	0.08	155.0	21.9
N	Stonehenge Fm.	2.68	0.13	128.8	23.9
O	Ledger Dolomite	2.83	0.13	181.6	33.8

Table 2: Multivariate regression results.

$r = 0.887$ Adjusted $R^2 = 0.729$
$F = 13.52$
$p < 0.0005$ Cutoff = 0.05

Variable	"t"	p	Cutoff
Intercept	-3.05	0.011	±0.025
L.A. Loss	-4.60	0.001	±0.025
Dry Density	-2.96	0.003	±0.025
Absorption	3.04	0.011	±0.025

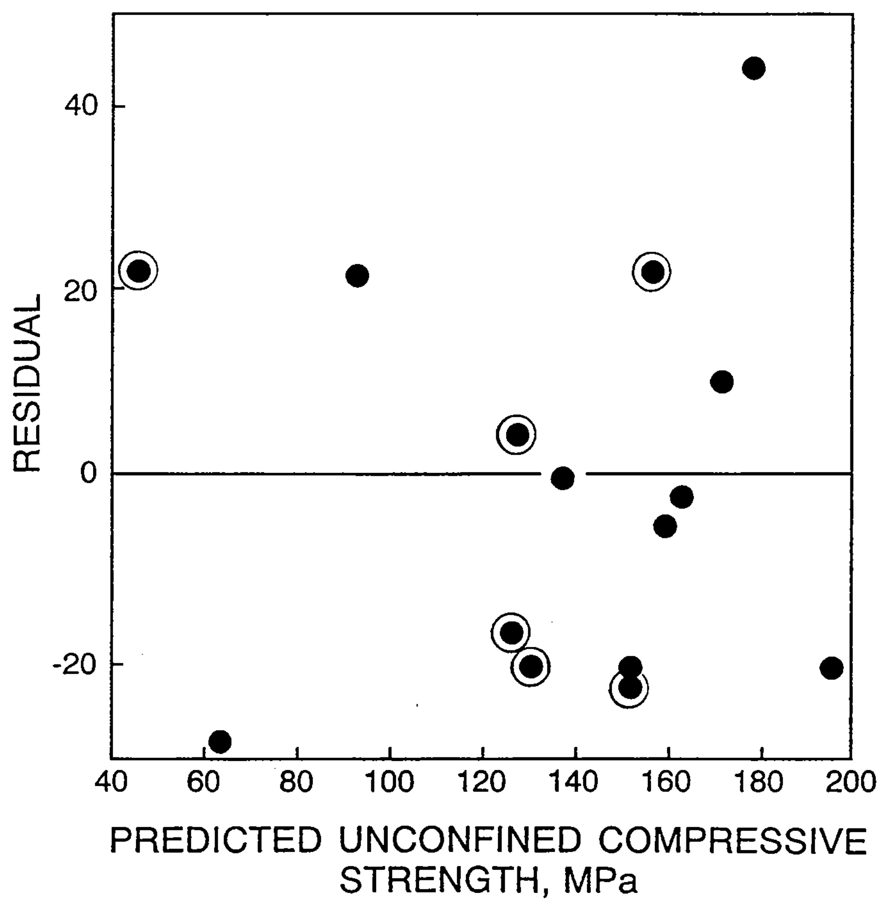


Figure 2. Plot of residual values vs. predicted values of unconfined compressive strength. Circled points represent those samples with less than 10 cores tested in unconfined compression.

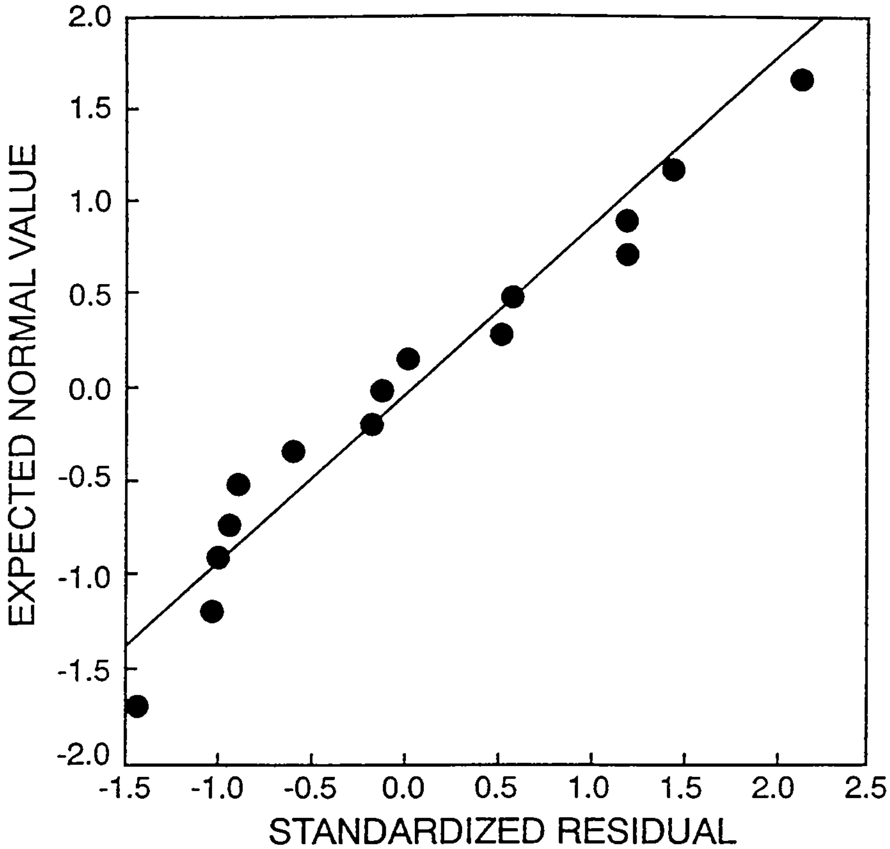


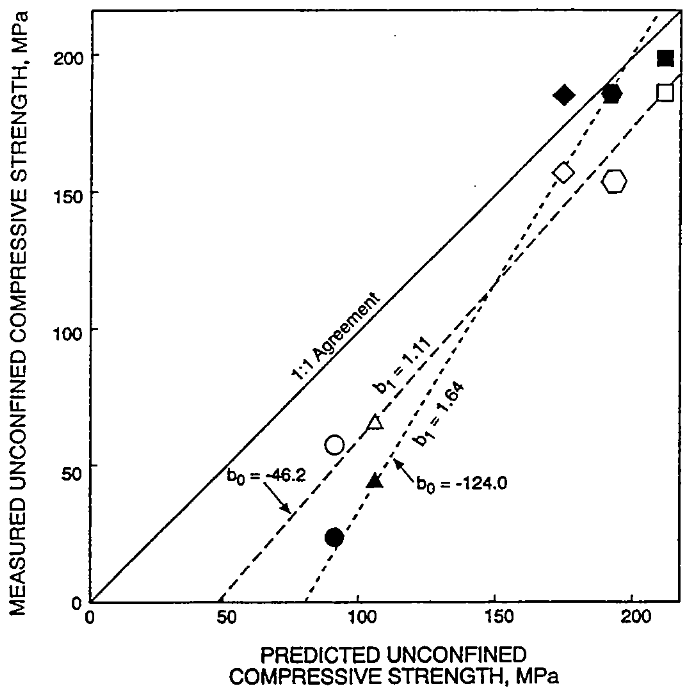
Figure 3. Normal probability plot for multivariate predictor of unconfined compressive strength.

VALIDATION OF PREDICTION EQUATION

In order to validate the previously developed prediction equation, five additional samples of carbonate rocks were tested for unconfined compressive strength, L.A. abrasion loss, dry density, and percent absorption and the average values were determined (Table 3). The L.A. abrasion loss, loss, dry density, and percent absorption data were used to predict the compressive strength values for the five rocks, using the prediction equation. The predicted values were then potted against the measured values as shown in Figure 4. At least three of the five data points lie close to the 1:1 line, indicating that the predictor equation is capable of reliably predicting compressive strength in at least 60 percent of the cases. It should be noted, however, that the two data points that do not fall close to the 1:1 line represent rocks with L.A. abrasion loss exceeding 40 percent, i.e., rocks not commonly used in engineering practice. If several more samples of carbonate rock with dry density, absorption, and particularly L.A. abrasion loss values falling within the range used to create the model were collected, a more robust assessment of the predictive capability of the model could be made.

Table 3: Unconfined compressive strength, L.A. abrasion loss, dry density, and absorption values for samples used in validation of multivariate model.

Formation	Unconfined Compressive Strength, MPa	L.A. Abrasion Loss, %	Dry Density g/cm^3	Absorption %
Sylagauga Marble	91	47	2.70	0.10
Cockeysville Marble	108	44	2.72	0.001
Middle Ordovician Limestone	192	24	2.77	0.002
Snyder Limestone	174	23	2.76	0.002
Onondaga Dolomite	210	18	2.50	1.99



SAMPLE	MULTIVARIATE	BIVARIATE	L.A. ABRASION, LOSS, %
Cockeysville Marble	▲	△	44
Sylagauga Marble	●	○	47
Onondaga Dolomite	■	□	18
Synder Limestone	◆	◇	23
Middle Ordovician Limestone	⬢	⬡	24

Figure 4. Plot of measured vs. predicted unconfined compressive strength.

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MITIGATING LANDSLIDE/ROCKFALL HAZARDS - A QUANTITATIVE EVALUATION OF RISK REDUCTION

by

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INTRODUCTION

An on-going urban transportation project in Pittsburgh, PA involves the construction of mitigation measures to reduce the hazards to the public of rockfalls and landslides on a historically active slope. The slope has an average slope of 1H:1V over a 300-400 ft change in elevation, and extends parallel to the roadway alignment for approximately 2 miles. In local areas the slope rises as much as 150 ft at a slope of 1H:3V. Through the process of a traditional qualitative approach to evaluating the rockfall and landslide hazards, a number of mitigation plans had been developed. The range of capital costs of these plans was considerable, as was the degree of construction difficulty. The question was posed as to how much construction work needs to be completed to provide adequate and acceptable standards of public safety.

One approach to addressing this problem is to use risk-based decision making techniques which are currently being widely encouraged by some government agencies. Risk-based decision making requires the decision-maker to get involved in the process of evaluating alternatives and coming up with recommendations. The decision maker (the risk-taker) contributes to the process by determining a set of project objectives and weighting the preferences among the objectives. The engineer can then rate each alternative against each objective and rank the alternatives, using the preference weightings, in decreasing order of their ability to maximize the suite of objectives.

However, for this project, it was decided first to establish the acceptability of the residual hazards in different zones along the alignment by attempting to quantify the risks and compare them with the hazard expectancy associated with engineering design in connection with other naturally occurring events (e.g., rainfall and earthquakes). With this information, cost estimates for each of the alternative design approaches, and the conclusions of the more traditional investigation and design approaches, a set of cost-effective mitigation measures could be identified and developed through final design.

Clearly, for a hypothetical random landslide or rockfall event, there is uncertainty about whether and how frequently a release will occur, uncertainty about whether debris will reach the roadway alignment given that the release occurs, and uncertainty in the nature and severity of the consequence if debris does reach the alignment. When faced with having to assess the possibility of an undesirable consequence of a design recommendation using traditional qualitative design approaches, it is very easy to let the multiplicity of uncertainties cloud the issues and thereby exponentially increase the apparent complexity of the problem. However, by deconvolving these uncertainties into small manageable parts using a probabilistic model, it becomes easier to deal with each of the small parts individually and then let the model combine the uncertainties in a rigorous manner to reconstruct the answer to the initial problem.

APPROACH

The simplest analytical form of the probabilistic model assumes that rockfall and landslide events at different locations along the slope are independent and random. Then, the expected number (average number) of landslide and rockfall events annually that lead to undesirable consequences for the roadway can be used as an approximation of the comparative performance of different remedial options. Furthermore, by deconvolving the problem into smaller parts, it also becomes a considerably easier task to calculate the total expected number of consequences of a given severity under existing conditions and for any given mitigation plan.

To make the problem manageable, the study area was divided into nine zones, each zone having certain defineable landslide characteristics. The results for each zone would be evaluated independently, just as one evaluates, for example, different bridges in a community independently. The expected annual number of consequences of different severity can be estimated from an analysis of the types of landslide/rockfall events (hereinafter referred collectively as landslide events), frequency of different types and sizes of landslide events, an assessment and analysis of the likelihood of landslide debris reaching the roadway alignment given an event, and an assessment of the likelihood of any one of a suite of different types of consequence given debris reaches the roadway. The expected annual number of consequences of a given severity in a given zone is simply equal to the sum, for all landslide types and size in that zone, of the products of the probability that debris from a random landslide event will reach the roadway alignment, the probability of the consequence that will result if debris reaches the alignment, and the frequency of such random landslide events.

The input parameters were assessed on the basis of a physical and photogrammetric survey of the slopes. The majority of the quantitative assessments were necessarily subjective, and to assist in this task, a formalized process of subjective probability self-assessments was carried out. Using this method, it is also possible to extend the approach to post construction conditions for a set of mitigation plans by changing the parameter assessments, and thereby, allow the order of magnitude benefits achieved by different construction costs to be compared.

SUMMARY OF GEOLOGIC CONDITIONS ON THE SLOPE

The geology of the slopes within the project area is within Pennsylvanian Age Conemaugh and Monongahela Groups, specifically within the upper Glenshaw and Lower Pittsburgh formations. Bedrock is relatively flat-lying sedimentary rock consisting of cyclic sequences of claystones, limestones, coals, shales, siltstones and sandstones. These rock units extend from the Pittsburgh redbed unit, including the Ames limestone, up through the Pittsburgh coal. Within this sequence, the Ames limestone, Grafton sandstone, Duquesne limestone, Birmingham shale, Morgantown sandstone, Connelsville sandstone and Pittsburgh limestone units are most notable and of particular importance as sources of rockfall activity. The differential weathering of the softer bedrock units, including the Pittsburgh redbeds, Harlem coal, Duquesne claystone and coal, Wellersburgh coal, Clarksburgh claystone and coal, and the claystones associated with the Pittsburgh limestone act as catalysts to undercut and activate the harder, more competent bedrock units noted above.

The slopes of study area are positioned adjacent to the Monongahela and Ohio Rivers and subsequently have experienced significant periods of intensive erosion during the downcutting of the rivers. This has tended to produce over steepened natural rock slopes and also increased the impact of valley stress relief on the slopes. These are very complex slopes to study and remediate because of the startigraphy, erosional episodes, slope heights and angels and past geologic history.

Due to the height and length of the slopes within the study area, the slope was broken down into smaller zones having discrete landslide and rockfall characteristics. These characteristics were determined through a review of historical aerial photographs, new oblique aerial photography analyses and features identified from on-slope geologic mapping. Zonation was based on the following criteria:

- position of the roadway alignment and grade with respect to the toe of the slope;
- the magnitude of rockfall hazard;
- exposures of primary and secondary rockfall generators and the height of the generator on the slope;
- similar failure types; and
- the conditions and angle of the slope as a result of past erosional events.

This zonation allowed for the evaluation of smaller, more focused study areas and the subsequent development of the qualitive and quantitative assessments.

MAKING SUBJECTIVE PROBABILITY ASSESSMENTS

Clearly, in the absence of a statistically representative data set, many of the factors involved in assessing the inputs to the quantitative risk assessment model will not be known with certainty. On these occasions, it is necessary to rely upon subjective assessments, based on judgment, consistent with all available information. In principle, the task of making subjective assessments should not be too demanding since everybody makes such assessments everyday. However, generally these assessments are not expressed explicitly, and consequently, difficulty may be encountered in defending the assessments to others. This difficulty may be largely overcome using a formalized process that has been proven effective in improving an individual's ability to assess uncertainty with greater accuracy (an individual's ability to assess his or her uncertainties accurately can be conditioned with training and practice).

An assessor's uncertainty is expressed by the breadth of his distribution (e.g., the difference between the 10% and 90% confidence levels). As the amount and relevance of the available information increases, the uncertainty should decrease. Since everyone's information base differs and because everyone would interpret that information differently, subjectively derived probability distributions are not unique and will vary among assessor's - there is no "true" probability distribution in such cases (for instance, "Experts" are simply those who have more knowledge about a specific topic than others, and they should have less uncertainty in their assessments within their area of expertise than those with little knowledge). However, this does not make subjective assessments any less valuable. If each person involved in making a specific group decision could input his assessment into a decision analysis procedure and conclude the same ranking of decision alternatives, each participant can both disagree with each others assessments but still reach consensus on the best strategy to follow. Ultimately, it is important to be able to defend that the best decision was made rather than which individual's point of view was correct. The intent of the subjective assessment is for the assessor to express his or her uncertainty accurately, properly reflecting the available information.

The formalized process of making subjective probability assessments (Roberds, 1990) is designed to make the self-assessor (as well as the users of such assessments) aware of the potential problems that can creep in to the assessments, so that they might be mitigated to the extent possible and their impacts on the validity of the assessment evaluated. The sensitivity of the intended application to the precision of the assessments can be evaluated numerically later during the analysis, and decisions affecting how much willingness or resources to apply to the assessment can then be based on the sensitivity study.

TYPE OF LANDSLIDE/ROCKFALL EVENT

Four broad types of landslide event were identified for inclusion in the quantitative risk assessment study:

- **Colluvium Slide** This involves reactivation of a large volume of soil, weathered rock, and vegetation by sliding along pre-existing failure surfaces. This type of landslide event, typically involving greater than 500 yd^3 of material, is often triggered by high precipitation conditions, removal of lateral support, and addition of fill. Once triggered, the landslide results in a flow, translation, or rotation of material that may travel considerable distances on certain sections of the slope, moving slowly or rapidly depending on moisture conditions, and can uproot trees, etc. However, this type of landslide event does not generally produce high point-impact forces.

- **Debris Avalanche** This involves the release of loose material, primarily talus (scree), rock fragments, and perhaps small amounts of colluvium, originating from more thinly bedded shale, sandstone, and limestone units. These events are triggered by a rockfall landing on a talus slope from above or giving way below a talus slope, heavy precipitation (washouts), or spring thaw (snow melt). The volume of material released is typically less than 500 yd^3. Once triggered, the material typically moves rapidly.

- **Rock Shower** This is the release of a number of small (typically less than 1/2 yd^3) blocks of rock, which then bounce and roll down the slope face in a shower, producing several moderate point-impact forces. This type of event is most often triggered by precipitation or thaw conditions, or removal of underlying support. The most typical sources of this type of rockfall hazard is the hard, jointed, thinly bedded, sandstone, sandy shale, and limestone.

- **Rock Fall** This is the release of a large (could exceed 10 yd^3) discrete block that rolls down the slope with great momentum. Point-impact forces could damage/destroy fences and structures. This kind of event is most often triggered by ice jacking, heavy precipitation, thaw, or removal of underlying support. The most likely sources of this type of rockfall hazard are the sandstone, sandy shales and limestone units. Rockfalls appear to be the most common failure type on the slopes above the Conrail shelf.

In the following discussion, "landslide" is used in a broad sense to include all four types of events. Each type of landslide was characterized by two different sizes in order to represent a range of possible hazard. The "large size" events were assumed to occur less frequently than "small size" events but had the potential for more devastating consequences. The actual size envisaged for large and small size events varied for each rock unit depending upon its geology and structure.

FREQUENCY OF LANDSLIDE EVENTS

The frequencies of landslide events were necessarily subjective and were based in part on limit historical data, and evidence of recent landslide activity on the slope. Assessments of the frequency of events were made for each landslide type and for each geologic unit on the slope in each zone. In addition, assessments of the reduction in the frequency events achieved by constructing certain mitigation measures were also made.

The result was a large number of point estimates representing the assessors' estimate for the most likely value. For instance, for one of the massive sandstone units in one of the high hazard zones, the frequency of a 350 yd^3 block being released under existing condtions was assessed as one in 100 years, which is consistent with the observation that one such size block lies on the railroad bench at the toe of the slope, and the railroad bench has been in existance for at least 150 years. At the same location, the frequency of 0.5 ft^3 blocks was 30 per year, which is again consistent with observations of the amount of debris on the slope at this location.

At this stage of the project, no effort has been made to evaluate the uncertainties in these estimates because until an initial result has been obtained form the risk model, it is not possible to evaluate whether the additional effort is justified or warranted.

LIKELIHOOD OF DEBRIS REACHING THE ROADWAY

Given that a landslide or rockfall event occurs anywhere on the slope it does not follow that the debris will reach the roadway. Several factors combine to prevent debris reaching the roadway alignment:

- Manmade structures ;
- Tree cover;
- Natural benching of the slope;
- Catch ditches of buffer zones

Remedial measures reduce rockfall hazards either by reducing the average frequency and size of rockfall events (e.g., by removing source material) as described previously, or by reducing the likelihood of rockfall debris reaching the roadway alignment (e.g., by obstructing the path of falling debris).

Assessments of the probability of a random landslide/rockfall reaching the alignment is complicated because landslides/rockfalls originate from different geologic units on the slope and with different recurrence frequencies from each unit. Furthermore, the engineer has few tools for quantitatively modeling the movement of landslide debris down slopes that usually contain a complex combination of features that impede progress. For "Rock Fall" events, computer simulation models (such as Colorado Rockfall Simulation Program) can be used to shed some light on the behavior of discrete falling boulders and the likelihood of a single boulder reaching the roadway. However, in most cases, the likelihood of landslide debris reaching the roadway has had to be assessed subjectively by those experienced in landslide behavior using methods that will be described later.

For a random landslide/rockfall event, the probability of debris reaching the roadway can be calculated from the sum, for all geologic units, of the relative likelihood that the event originated from a particular unit and the assessed probability that a single landslide/rockfall event of that type and size and originating from that unit will reach the roadway. This calculation is illustrated in the probability tree presented in Figure 1. By aggregating the likelihood of debris from a landslide from each of the units as a separate step to the risk model itself, the risk model itself is simplified.

CONSEQUENCES OF LANDSLIDE EVENTS

Once landslide/rockfall debris reaches the roadway alignment, some kind of consequence will be incurred. For this purpose the range of possible consequences has been divided into three categories:

Geologic Unit	Frequency of Landslide Events	Relative Probability of a Random Landslide Being From Unit	Probability of Debris Reaching Roadway Given Fall From Unit	Probability of Debris Reaching Roadway
Unit 1	F1	$P'1 = \frac{F1}{F1+F2+....+Fn}$	P"1	P'1 P"1
Unit 2	F2	$P'2 = \frac{F2}{F1+F2+....+Fn}$	P"2	P'2 P"2
Unit 3	F3	$P'3 = \frac{F3}{F1+F2+....+Fn}$	P"3	P'3 P"3
Unit n	Fn	$P'n = \frac{Fn}{F1+F2+....+Fn}$	P"n	P'n P"n
Frequency from all units =	$\sum_{i=1}^{i=n} Fi$		Total =	$\sum_{i=1}^{i=n} P'i\,P''i$

Figure 1
Calculation of Probability of Debris Reaching the Roadway

- **Minor**. The landslide results in no damage or shutdown of the roadway for cleanup. The debris is caught in ditch or at the side of the roadway, and can be cleaned up under the routine operations and maintenance activities.

- **Moderate**. The landslide results in material obstructing the roadway or inflicting minor damage to structures or property. Immediate clean-up is required, but is limited to clearance of debris from the roadway and ditch and minor repairs to retention fences or other structures. This work may shut the roadway down for several hours; and

- **Severe**. The landslide results in a shutdown of roadway operations for more than several hours to evaluate the causes and possible recurrence of such an event, clear large quantities of debris, reinstate roadway utilities and right-of-way, and make repairs to seriously damaged structures. A sever consequence may or may not involve damage to property and/or personal injury or loss of life.

The probability that landslide debris reaching the roadway alignment will lead to one of the three types of consequences described above was also assessed subjectively. Factors that were evaluated in these assessments included the size and speed of debris movement; the geometry of the slope at the toe and the proximity of the roadway alignment; and the anticipated traffic volume. For instance, with any of the landslide events involving large quantities of material it is envisaged that some kind of engineering evaluation will be required prior to re-opening the roadway to traffic, irrespective of property damage or personal injury, and this will tend to mean a significantly higher likelihood of a severe consequence by definition. In the case of large rockfalls and rock showers, it was assessed that the likelihood of such an evaluation taking more than a day is nine times as likely as it taking less (e.g., likelihood of a minor consequence is 10%). With regard to the event itself, the blocks of rock would likely be moving quickly, and if they were to impact a structure, would likely inflict considerable damage. The likelihood of moving boulders impacting a moving vehicle is considered small because the traffic is expected to be small and the passage of the boulders onto and across the roadway occurs within a short space of time. As with small size events, the main hazard will be buses hitting stationary boulders. Given the size of the boulders, it is assumed that drivers would recognize the obstruction earlier than a small boulder, but that impact would more likely lead to serious damage to the bus and its occupants. Therefore, a severe consequence is considered twice as likely as a moderate consequence.

REMEDIAL STRATEGIES

As stated perviously, mitigation measures reduce landslide hazards either by reducing the average frequency and size of events (e.g., by removing source material) or by reducing the likelihood of debris reaching the alignment (e.g., by obstructing the path of falling debris). Once debris reaches the roadway alignment, the consequences remain the same. Simply implementing remedial measures does little to affect the consequences of landslides, although some kinds of measures can alter the likelihood (e.g., warning systems).

Three remedial strategies were initially devised for each zone that represent a broad spectrum of reasonably feasible measures that could be implemented to reduce landslide/rockfall hazards:

- **No Action** - i.e., leave conditions as existing;
- **Toe Controls** - e.g., rockfall fences; and
- **Toe Controls and Slope Treatments** - e.g., rockfall fences plus tieback large blocks, bolts and mesh protection of near-vertical faces, etc. in various combinations to suit zone conditions

The No Action alternative is always required in a quantitative risk assessment to provide a baseline with which to compare the benefits of other alternatives. Adding slope treatments to toe controls increases the cost of this remedial alternative, but at the same time reduces the risks by reducing the frequency of landslide events. Other solutions, such as laying the slope back or choosing a new alignment could have been included, but were not considered realistic options for consideration at this time. It was decided that these options would only be evaluated if none of the mitigation measures listed above would adequately safeguard public safety.

RISK ASSESSMENT RESULTS

As stated earlier, the average or expected number of annual consequences of a given type can be estimated for a large number of independent random events by summing for all landslide types and size along the slope, the products of the probability that debris from a random landslide event will reach the roadway alignment, the probability of the consequence that will result if debris reaches the alignment, and the frequency of such random landslide events. This calculation is illustrated in the probability tree presented in Figure 1.

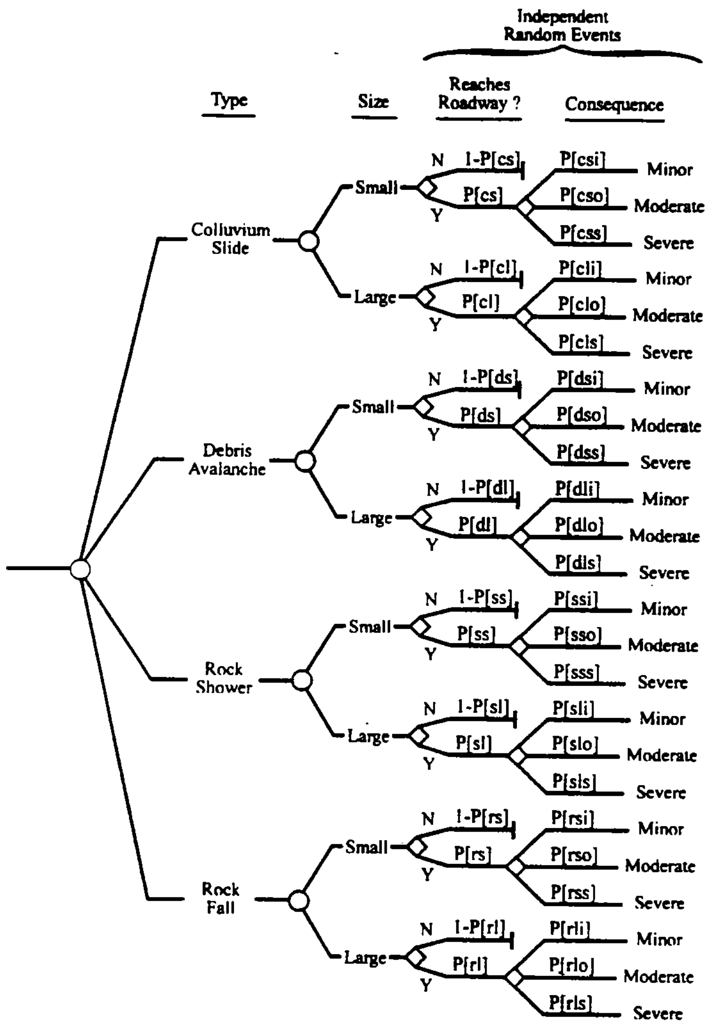
The results of the initial risk assessment are summarized in Table 1. The results are dominated by the high frequency of small rock fall events, and the high likelihood of such events reaching the alignment in zones where the roadway alignment is close to the toe of the slope. The average annual number of consequences of any category along the entire length of the slope is 20, which is typical for an average winter in the area. Of significance is the average annual number of severe consequences along the entire slope under the existing conditions (0.1/yr). Using this result, the probability of observing the number of severe consequences (2) recorded between 1937 and 1952 in the vicinity of the slope is 25%, and the probability of observing two or more severe consequences is 44%. Both these result provide reasonable verification of the validity of the subjective assessments that were made for the quantitative risk assessment.

EVALUATION OF ALTERNATIVE REMEDIAL STRATEGIES

When the analysis is extended to the expected performance of the alternative remedial strategies under consideration, the results show that a significant reduction in the rockfall hazard can be achieved by simply implementing toe controls. Overall the total number of incidents is expected to reduce by an order of magnitude. Of most significance is the reduction in the number of annual moderate and severe consequences which are reduced from 0.62/yr to 0.18/yr for moderate consequences over the entire length of the study area, and from 0.15/yr to 0.04/yr for severe consequences (i.e., a four-fold reduction in the hazard). If one further applies slope treatments to the most hazardous zones as suggested by the qualitative risk assessment (e.g., to Zones 2, 3 and 8), the total number of annual moderate and severe consequences can be reduced by a further 50%, approximately, to 0.09/yr and 0.02/yr. For the most hazardous rockfall zone, Zone 8, the average number of annual moderate and severe consequences can only be reduced to 0.08/yr and 0.02/yr respectively using the scope of mitigation measures included in this study, and thus the hazards in this zone dominate the overall risk result. Clearly, this is an area requiring more design effort as the project proceeds.

COMPARISON WITH COMMON DESIGN STANDARDS

For this project, the approach taken to evaluate the acceptability of the alternative mitigation measures to protect users of the roadway from landslide hazards, is to compare the residual hazards in Table 1 after constructing a set of mitigation measures with the hazard expectancy associated with engineering design in connection with other naturally occurring events (e.g., rainfall and earthquakes). In comparing the results of the quantitative landslide risk assessment described above, it is necessary to evaluate the hazards within each zone independently in the same way that



Path Prob.	Annual Freqency of Events of Type and Size	Expected Annual Number of Consequences
P[cs]P[csi]		P[cs]P[csi]F(cs)
P[cs]P[cso]	F(cs)	P[cs]P[cso]F(cs)
P[cs]P[css]		P[cs]P[css]F(cs)
P[cl]P[cli]		P[cl]P[cli]F(cl)
P[cl]P[clo]	F(cl)	P[cl]P[clo]F(cl)
P[cl]P[cls]		P[cl]P[cls]F(cl)
P[ds]P[dsi]		P[ds]P[dsi]F(ds)
P[ds]P[dso]	F(ds)	P[ds]P[dso]F(ds)
P[ds]P[dss]		P[ds]P[dss]F(ds)
P[dl]P[dli]		P[dl]P[dli]F(dl)
P[dl]P[dlo]	F(dl)	P[dl]P[dlo]F(dl)
P[dl]P[dls]		P[dl]P[dls]F(dl)
P[ss]P[ssi]		P[ss]P[ssi]F(ss)
P[ss]P[sso]	F(ss)	P[ss]P[sso]F(ss)
P[ss]P[sss]		P[ss]P[sss]F(ss)
P[sl]P[sli]		P[sl]P[sli]F(sl)
P[sl]P[slo]	F(sl)	P[sl]P[slo]F(sl)
P[sl]P[sls]		P[sl]P[sls]F(sl)
P[rs]P[rsi]		P[rs]P[rsi]F(rs)
P[rs]P[rso]	F(rs)	P[rs]P[rso]F(rs)
P[rs]P[rss]		P[rs]P[rss]F(rs)
P[rl]P[rli]		P[rl]P[rli]F(rl)
P[rl]P[rlo]	F(rl)	P[rl]P[rlo]F(rl)
P[rl]P[rls]		P[rl]P[rls]F(rl)

Figure 2
Risk Assessment Model for a Single Zone on the Slope and a Single Remedial Strategy

one evaluates bridges in a community independently. In this case, with the exception of Zone 8, each of the other zones has an annual expected number of moderate and severe consequences of less than 0.01 and 0.002.

In the case of rainfall events, drainage channels and culverts in Pennsylvania are often designed for a 10-year rainfall event. The 10-year rainfall event is of a magnitude that would expect to be exceeded once in every 10 years on average (average annual frequency = 0.1/yr). In the case of a culvert passing beneath a low use public road (0-400 average daily traffic), this means that the capacity of the culvert would expect to be exceeded and the road flooded once every 10 years on average. Such an event would likely shut the road down for several hours, and may require some repair work to the roadway and embankments after the storm had passed (i.e., the consequence is

TABLE 1

Summary of Average Landslide Risks

Slope Zone	Remedial Strategy	Total Annual Expected Number of Consequences (All Types)	Total Annual Expected Number of Minor Consequences	Total Annual Expected Number of Moderate Consequences	Total Annual Expected Number of Severe Consequences
1	Existing Conditions	8 E-02	4 E-02	3 E-02	8 E-03
	Toe Controls	2 E-03	1 E-03	8 E-04	2 E-04
	Toe Controls + Slope Measures	4 E-04	2 E-04	2 E-04	4 E-05
2	Existing Conditions	5 E-02	3 E-02	2 E-02	4 E-03
	Toe Controls	3 E-03	1 E-03	1 E-03	3 E-04
	Toe Controls + Slope Measures	6 E-04	3 E-04	2 E-04	6 E-05
3	Existing Conditions	4 E-02	2 E-02	2 E-02	4 E-03
	Toe Controls	4 E-03	3 E-03	1 E-03	3 E-04
	Toe Controls + Slope Measures	2 E-03	1 E-03	2 E-04	6 E-05
4	Existing Conditions	6 E+00	6 E+00	8 E-02	2 E-02
	Toe Controls	2 E-02	1 E-02	6 E-03	1 E-03
	Toe Controls + Slope Measures	9 E-03	6 E-03	2 E-03	6 E-04
5	Existing Conditions	1 E-03	9 E-04	1 E-04	1 E-05
	Toe Controls	1 E-03	9 E-04	1 E-04	1 E-05
	Toe Controls + Slope Measures	1 E-03	9 E-04	1 E-04	1 E-05
6	Existing Conditions	0 E+00	0 E+00	0 E+00	0 E+00
	Toe Controls	0 E+00	0 E+00	0 E+00	0 E+00
	Toe Controls + Slope Measures	0 E+00	0 E+00	0 E+00	0 E+00
7	Existing Conditions	1 E+00	1 E+00	6 E-02	1 E-02
	Toe Controls	6 E-03	4 E-03	1 E-03	3 E-04
	Toe Controls + Slope Measures	6 E-03	4 E-03	1 E-03	3 E-04
8	Existing Conditions	1 E+01	1 E+01	4 E-01	9 E-02
	Toe Controls	4 E+00	4 E+00	2 E-01	4 E-02
	Toe Controls + Slope Measures	1 E+00	1 E+00	8 E-02	2 E-02
9	Existing Conditions	3 E+00	3 E+00	5 E-02	1 E-02
	Toe Controls	2 E-01	2 E-01	1 E-02	2 E-03
	Toe Controls + Slope Measures	2 E-01	2 E-01	1 E-02	2 E-03
All Zones	Existing Conditions	2 E+01	2 E+01	7 E-01	1 E-01
	Toe Controls	5 E+00	4 E+00	2 E-01	4 E-02
	Toe Controls + Slope Measures	2 E+00	2 E+00	1 E-01	2 E-02

equivalent to a moderate consequence described for this study). In the case of high-usage roads (>5,000 average daily traffic), a culvert beneath the road might be designed for a 50-year rainfall event (average annual frequency of 0.02/yr), since the consequence of exceeding the capacity of the culvert is likely to be more severe.

It is more difficult to make such analogies for structures such as dams because the magnitude of the rainfall event used to size the spillway is not usually based on storm frequencies. Only for small dams in rural areas, where failure might damage farm buildings, agricultural land, or township or country roads, without bodily injury or loss of life, is a storm frequency (100-year storm) used to size the emergency spillway. For all other dams, the Probable Maximum Precipitation (PMP) is used as an input to determining the design capacity of the emergency spillway and freeboard. The PMP is defined as the reasonable maximization of the meteorological factors that operate to produce a maximum storm. The PMP has a low, but unknown, probability of occurrence. Those that have analyzed PMPs in terms of average annual frequencies suggest results in the range of 10^{-11} to 10^{-19} (ASCE, 1988) which is consistent with the philosophy of seeking a zero risk. If this was to be used as a design standard for the study slope, the only landslide mitigation solution applicable to the roadway would be to lay the entire slope back or find another alignment away from the toe of the slope.

One must note that in the case of severe rainfall events, meteorologists can give advance warning of a certain magnitude event moving into an area, and this gives communities a chance to evacuate safely or start to implement temporary protection measures for property. So it can be argued that the PMP safety standard is overly severe compared with other events for which no warning is given (e.g.. seismic events),

In the case of seismic events (earthquakes), there also exist multiple standards. In the case of landfills, the United States Environmental Protection Agency has recently promulgated new design standards that require landfills to be designed for a seismic event having a 10% chance of being exceeded in 250 years (average annual exceedence frequency of 0.0004) which is extremely low and reflects the longer design life expectancy of landfills by comparison with roadway structures, for instance, and the current level of public perceptions and concern with regard to environmental disasters. In the case of structures such as bridges, AASHTO has adopted the 1986 printing of the Seismic Design Guidelines for Roadway Bridges published by the Applied Technology Council (funded by FHWA), which suggest that bridges be designed for a seismic event having a 10% chance of being exceeded in 50 years. Given that it is usually assumed that the recurrence of seismic events is represented by a Poisson process, this translates to an average annual exceedence frequency of 0.002/yr.

The consequences of a bridge collapse during a seismic event are very similar to the consequences of a rockfall (e.g., significant disruption and rerouting of traffic; major repairs and investigations; possible damage to property; etc.). Therefore, this design standard is possibly more applicable to the mitigation of rockfalls on the study slope. In all zones with the exception of Zone 8, this design standard is achieved in Zones 5 and 6 under existing conditions, and is achieved in all zones with the exception of Zone 8 using simple toe controls. The results for Zones 4 and 9 are close to the design standard, and given the expected range of uncertainty in the assessments, it would be prudent to implement slope measures in these zones as well. In Zone 8, the residual risks are unnacceptable even if toe controls are supplemented with the scope of limited slope measures considered. Clearly, it is necessary to pay further attention to Zone 8 and seek further ways of reducing the apparent residual hazards.

These average annual frequencies are small and perhaps difficult to interpret in terms of how safe is safe enough, and how close are the comparisons among the design standards. Another, and possibly more sensitive, way of looking at these standards is in terms of the probability that a certain number of exceedences might occur in a period of interest. Such a translation assumes that the events, be they landslide, rainfall, or seismic, are represented by a Poisson process: events occur randomly and independently; and the average annual frequency for a given size event is a constant. If, for example, one assumes that toe controls are implemented in all zones and slope measures are implemented in Zones 1, 2, 3, 4 and 8, a comparison of the residual landslide risks against the residual risks for culverst designed to withstand rainfall events and bridges designed to withstand seismic events is provided in Table 2. Again this table shows the degree to which the proposed suite of landslide mitigation measures provides standards of public safety that are at least as protective as commonly adopted design standards for bridges

CONCLUSIONS

This paper has summarized a quantitative study of the risks to the alignment of an on-going transportation project in Pittsburgh, PA associated with landslides on the adjacent slopes. The quantitative approach to risk assessment has proved to be a useful tool and a valuable first step in understanding and supporting the results of more traditional qualitative risk assessment methods. It does not always follow that by deconvolving the problem into smaller parts, making independent assessments of these smaller parts, and then combining these assessments to characterize the overall risk, that the result will support a qualitative risk assessment. When it doesn't, the details of the quantitative approach can be used to assist in either identifying what sensitive parameters might have been overlooked or underestimated in the qualitative risk assessment, or understanding what changes to the initial assessments would have to be made so that the result of the quantitative assessment become consistent with the result of the qualitative assessment.

It is this ability to scrutinize what factors contribute to the risks and affect design decisions that characterizes the valuable role of quantitative risk assessments in complex design projects such as this. In this case, the results highlighted that the remaining hazards in Zone 8 keep the residual risks higher than those normally associated with the design of other structures exposed to natural hazards (e.g., bridges and culverts). It is, therefore, necessary to focus attention on this zone during the final design phase. Ultimately, final recommendations on what mitigation measures should be constructed in each zone will be based also on the results of the more traditional qualitative design methods and the cost-effectiveness of the proposed mitigation plans.

TABLE 2

Summary of Risks for Moderate and Severe Consequences

Consequence	Event	Average Annual Frequency (/yr)	Likelihood of Exceedence in 50 years			
			1 occurrence	2 occurrences	5 occurrences	10 occurrences
Moderate	Rainfall (culvert)	1.00E-01	99.3%	96.0%	56.0%	3.2%
	Landslide Zone 1	8.00E-04	1.0%	---	---	---
	Landslide Zone 2	2.00E-04	1.0%	---	---	---
	Landslide Zone 3	2.00E-04	1.0%	---	---	---
	Landslide Zone 4	6.00E-03	9.5%	0.4%	---	---
	Landslide Zone 5	1.00E-04	0.5%	---	---	---
	Landslide Zone 6	0	---	---	---	---
	Landslide Zone 7	1.00E-03	4.8%	---	---	---
	Landslide Zone 8	8.00E-02	98.2%	90.9%	37.1%	0.8%
	Landslide Zone 9	1.00E-02	39.4%	9.0%	---	---
Severe	Seismic (bridges)	2.00E-03	9.5%	0.4%	---	---
	Landslide Zone 1	2.00E-04	0.2%	---	---	---
	Landslide Zone 2	6.00E-05	0.3%	---	---	---
	Landslide Zone 3	6.00E-05	0.3%	---	---	---
	Landslide Zone 4	1.00E-03	4.9%	---	---	---
	Landslide Zone 5	1.00E-05	---	---	---	---
	Landslide Zone 6	0	---	---	---	---
	Landslide Zone 7	3.00E-04	1.5%	---	---	---
	Landslide Zone 8	2.00E-02	63.2%	26.4%	0.3%	---
	Landslide Zone 9	2.00E-03	9.5%	0.4%	---	---

Note: --- means that the likelihood is less than 0.2%

REFERENCES

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