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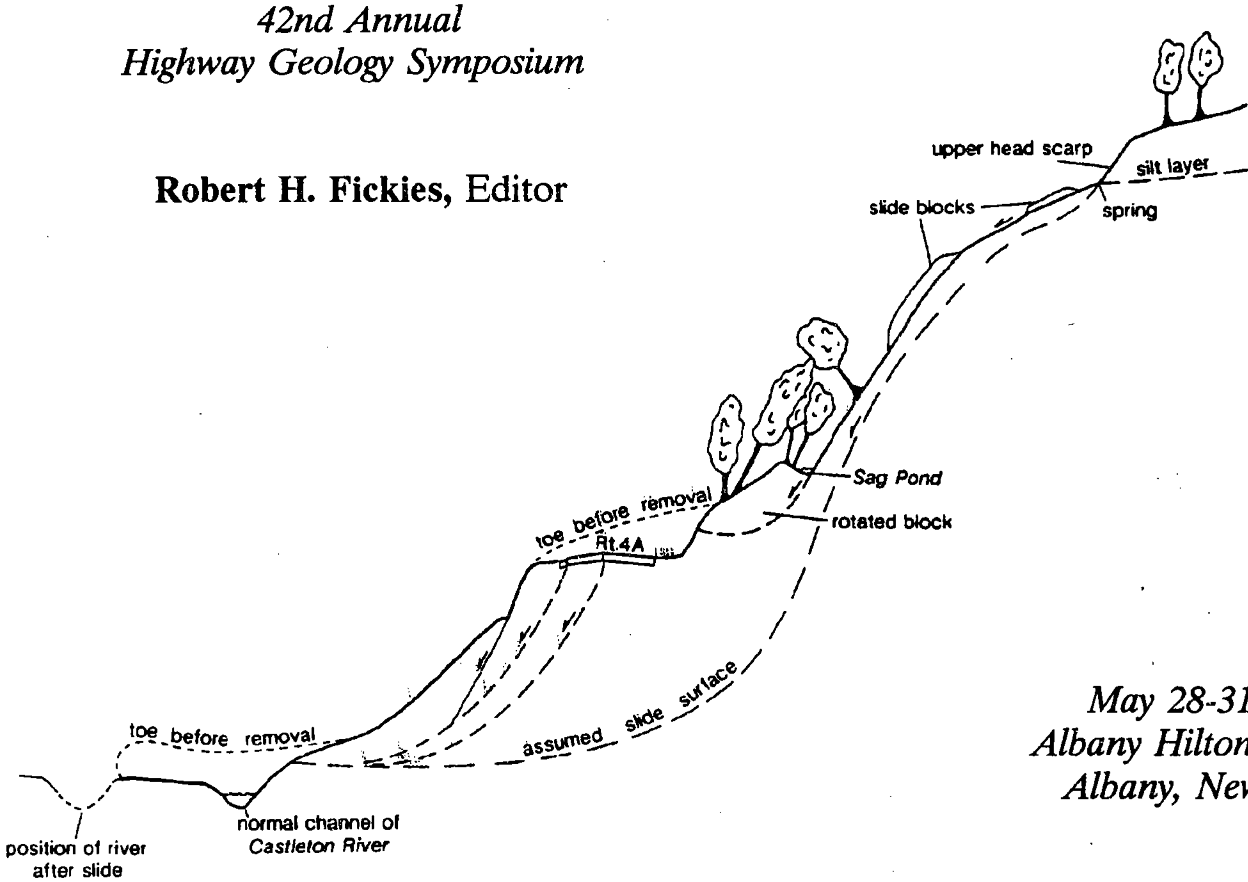
"Geologic Complexities in the Highway Environment"

Proceedings of the
42nd Annual
Highway Geology Symposium

Robert H. Fickies, Editor



May 28-31, 1991
Albany Hilton Hotel
Albany, New York

New York State Department of Transportation

MARIO M. QUOMO, Governor
FRANKLIN E. WHITE, Commissioner

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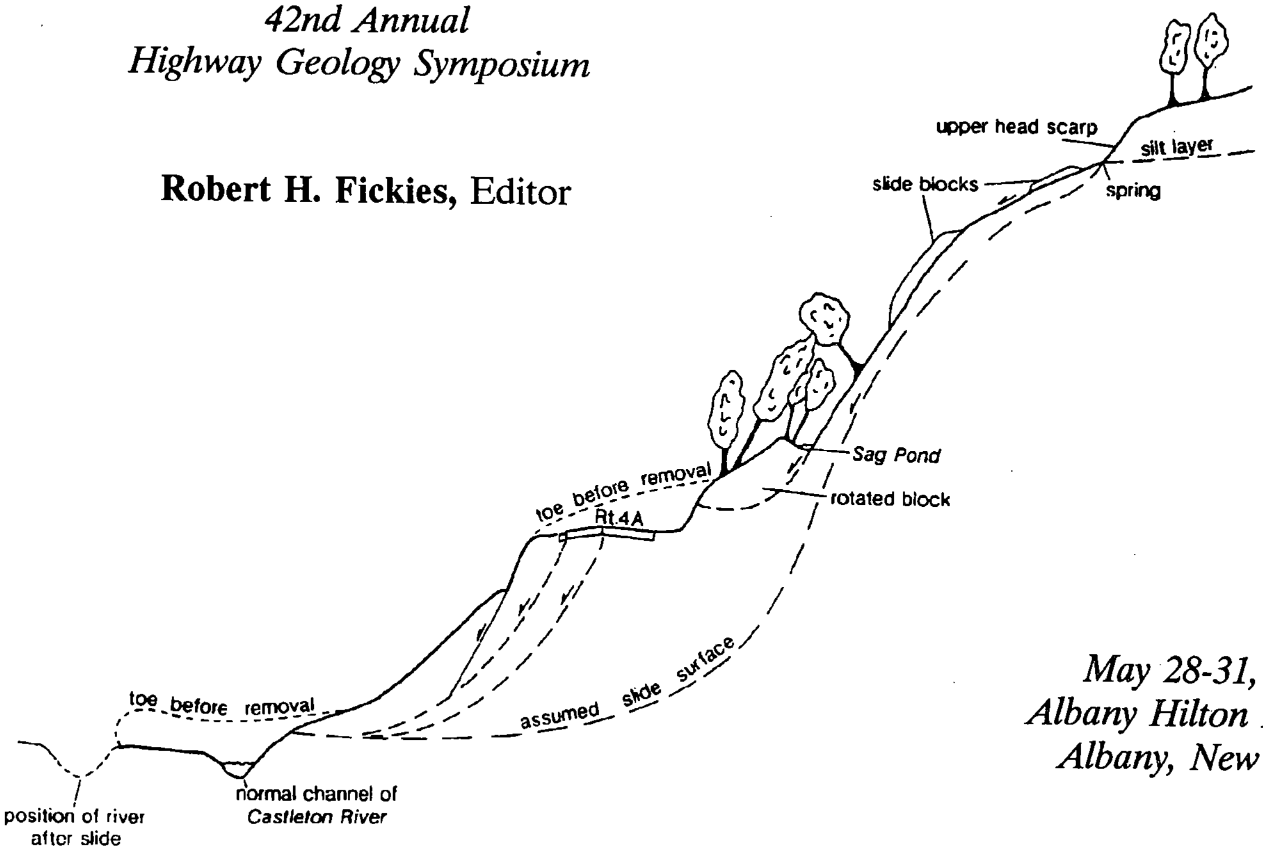

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co-sponsored by

American Society of Civil Engineers•American Institute of Professional Geologists
New York State Department of Transportation•New York State Geological Survey

April, 1993

42nd PROCEEDINGS VOLUME, HIGHWAY GEOLOGY SYMPOSIUM DEDICATED TO BURRELL S. WHITLOW (1929-1990)

Burrell Stewart Whitlow was born in Vinton, Virginia in 1929. He was educated at the Virginia Military Institute where he received a bachelor of science degree in civil engineering, in 1951.

Burrell Whitlow served in the U.S. Army in Iceland from 1951 to 1953, attaining the rank of captain. Following his return to the United States, he constructed bridges for the Commonwealth of Virginia for a year, following which he spent several years in graduate school at Virginia Tech, studying geology and geophysics. In 1956, he joined Hayes, Seay, Mattern & Mattern in Roanoke, and eventually rose to the position of senior associate. In 1972, Mr. Whitlow became founder and president of Geotechnics Inc., a geotechnical consulting firm in Roanoke.

Mr. Whitlow was a Registered Professional Geologist and Certified Professional Geologist, a member of the American Institute of Professional Geologists and the American Society of Civil Engineers, and a former president of the Virginia Section of the American Institute of Professional Geologists.

Burrell Whitlow was an active member of the Highway Geology Symposium, being involved during the 1950's, contributing to the origination of the symposium. He received the H.G.S. National Medallion Award in 1978, and later served as Vice Chairman of the Steering Committee in 1979 and 1980.

Several of Burrell Whitlow's more memorable technical papers include, "*A Return to Reason - The Application of Simple Geology to Complex Urban Problems,*" "*The General Practice of Engineering Geology,*" and "*The Investigation of Deterioration in Concrete Roadway Slab of the Robert E. Lee Bridge, Richmond, Virginia.*"

He was the National Academy of Sciences Representative for a 1968 Lecture Tour of Foreign Countries, which included the Soviet Union, Denmark, Poland, and Czechoslovakia. He had a harrowing experience on the morning of August 21st, while in Prague with a group of about 4,500 geologists. As related by Jerry Eggleston in the Virginia Military Institute Alumni Review, "He awoke to the sound of low-flying planes and looked out the window and mumbled, "Oh, nyet" when he saw and heard tanks clanking along the streets. The Russians weren't coming - they were there. Whit had been invaded!" Two days later, he escaped by crossing the Czech Border into West Germany.

Burrell Whitlow was known for his uncompromising character. He never jeopardized his personal honor and integrity regardless of the situation or the potential profits. Burrell also believed very strongly in his duty and responsibility to his family. Mr. Burrell Whitlow passed away at his home in Vinton, Virginia, on October 11, 1990.

Harry Moore

42nd Annual Highway Geology Symposium
Program Agenda

TUESDAY, 28 MAY, 1991

Geotechnical Borings - Sponsored by the Transportation Research Board in conjunction with the 42nd Highway Geology Symposium.

10:00 AM to 5:00 PM - Exhibitor Display
6:00 PM to 9:00 PM - Exhibitor's Night and Registration

WEDNESDAY, 29 MAY, 1991

8:00 AM to 5:00 PM - Registration
10:00 AM to 5:00 PM - Exhibitor Display

8:30 AM Welcome: Harry Moore, Chairman HGS Steering Committee
8:40 AM Opening Remarks:
Verne C. McGuffey, Chairman
42nd Highway Geology Symposium

8:50 AM Keynote Address: "Geology of New York"
Dr. Robert Fakundiny, State Geologist;
New York State Geological Survey

10:30 AM to 12 Noon

TECHNICAL SESSION I

Geologic Innovations on the Pennsylvania Turnpike
B. Bydlon; Pennsylvania Turnpike Commission

Seismic Refraction Technique Applied to Highway Design in a Strip Mined Area of Southwestern Pennsylvania
D. Rudenko, H. Ackermann; Vibra-Tech Engineers
W. Lawrence; Geomechanics

Evaluation of Acid Leachate Potential in Highway Construction
M. Decker, G. Jacobsen; Haley & Aldrich

Quarry Layers - Stratigraphic Units that Serve the Public Interest
S. Stokowski; Stone Products Consultants

12 Noon to 1:30 PM Lunch

TECHNICAL SESSION II

Roadways in Karst Terrane
J. Fischer, R. Greene; Geoscience Services

Highway Construction in Karst Terranes: Avoiding and Remediating Collapse Features
J. Mellett; New York University
B. Maccarillo; New Jersey DOT

Application of Non-destructive Testing Techniques to Slope Stability and Sinkhole Monitoring
H. R. Hardy; Pennsylvania State University

Rock Slope Excavation and Stabilization Methods in Highway Construction: Interstate 287 Extension, New Jersey
S. Brandon; Golder Associates

3:20 PM to 5:00 PM

Rock Slope Investigations at Selected Hudson Valley Sites
L. Hale; Dunn Geoscience Engineering Co., P.C.

Rock Slope Inventory, Evaluation and Remediation for Sections Along the New York State Thruway
J. Burke, S. LeFevre; Clough Harbour & Associates

A Design for a Temporary Reusable Rock Catchment Barrier
R. Cross; New York State Thruway Authority

A Field Trip Review
C. Bolton; New York State DOT

THURSDAY, 30 MAY, 1991

Field Trip to West Point Area, including a stop on the New York State Thruway.

6:00 PM to 7:00 PM Social Hour

7:00 PM to 9:30 PM Banquet

Speaker - Anson Piper; Adirondack Community College

FRIDAY, 31 MAY, 1991

TECHNICAL SESSION III

8:00 AM to 9:40 AM

Complex Geology at Complex Sites
L. Abramson; Parsons, Brinkerhoff, Quade & Douglas

Treated Aggregate in an Asphaltic Concrete Road: An Apparent Success
J. Dunn, G. Banino; Dunn Geoscience Corporation
D. LaGrand; General Electric Co.

Improving Aggregate Quality by Chemical Treatment
P. Hudec, F. Achampong; University of Windsor

Highway Bridge Failure by Foundation Scour and Instability
A. Parola, D. Hagerty; University of Louisville

10:00 AM to 12 Noon

Measurement of Scour at Selected Bridges in NY
G. Butch; USGS Water Resources Division

Ground Penetration Radar Study of Riverbed Scour in New York State
W. Horne; Clarkson University

Clues to Landslide Identification and Investigation
V. McGuffey; New York State DOT

Slope Failure Probability for Mixed Layer Soils
S. Thornton; University of Arkansas
S. Garnett; Grubbs, Garner & Hoskyn

Northern New England Landslides
C. Baskerville; Central Connecticut State University
G. Ohlmacher; Mary Washington College

HIGHWAY GEOLOGY SYMPOSIUM
NATIONAL STEERING COMMITTEE

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Richard H. Cross, New York State Thruway Authority

Mark Dore, New York State Office of General Services

Robert H. Fickies, New York State Geological Survey

Stephen E. Sweeney, New York State Department of Transportation

Table of Contents

Geotechnical Innovations on the Pennsylvania Turnpike

By Bernard T. Bydlon
Assistant Program Manager - East
Pennsylvania Turnpike Commission

INTRODUCTION

America's first superhighway, the Pennsylvania Turnpike, opened its original section, from Carlisle to Irwin, in October 1940. The Turnpike expanded in November 1950, with the opening of the Philadelphia Extension, from Carlisle to Valley Forge. A Western Extension from Irwin to the Ohio state line was added in December 1951, expanding the Turnpike. The Delaware River Extension, from Valley Forge to the Delaware River, opened in November 1954, and the Northeast Extension, from Norristown to Scranton, was opened in November, 1957.[1]

Currently, celebrating its 50th year in service, the Turnpike is conducting a vast rehabilitation and expansion program. Many previously undisclosed geological complexities for the construction industry have been encountered in this program.[2]

The Pennsylvania Turnpike has used new construction technology. For example, geogrids allow widening of highway slopes when limitations exist on final slope angle and right-of-way. The Turnpike has also used gabion mattresses and a geoweb product to stabilize existing slopes under bridges. In addition, the Turnpike has made extensive use of reinforced earth walls and post-and-plank walls to widen the existing roadway with minimal property acquisition or slope buildup in areas of limited right-of-way.

A new Lehigh Tunnel is under construction in northeastern Pennsylvania, 13 miles north of Allentown. This is the first vehicular tunnel to be constructed in the United States by means of the New Austrian Tunneling Method. Soil nailing is being used at the north tunnel approach.

Rock cuts present difficult problems in construction and maintenance. The Turnpike has undertaken a system-wide study of all rock cuts and classified them on the basis of need for repair. The Turnpike has undertaken feasibility studies, preliminary designs, final designs and completed construction on certain cuts. Factors to be considered include existing roadway, existing right-of-way, traffic volumes, construction time, and construction cost. Each cut must be evaluated differently based on these factors and each cut presents site-specific geological problems as well.

GENERAL

Geoweb is a web of material that looks similar to a honeycomb. The honeycomb is backfilled with stone about the size of subbase material. The Turnpike used Geoweb as an experimental product to protect bridge abutment slopes, and it proved successful. Another product, gabion mattress, has been used very successfully to protect bridge abutment slopes and also in stream diversions.

As part of an accelerated construction program, certain sections of the existing roadway, or shoulders had to be widened. The widening necessary ranged from 2" to 16". Sections involved included in this widening were bridge structures, cut sections, and fill sections.

In areas where sufficient right-of-way was available, cut sections were laid back to a satisfactory slope angle. In areas of limited right-of-way, an alternative method was needed. The method used most on the Pennsylvania Turnpike is post-and-plank walls (see Figure 1). A soil nailing wall was used on the Lehigh Tunnel No. 2 project. This procedure allowed for a steeper slope angle without excessive excavation at the tunnel's north portal.

Fill sections that required widening were built up by

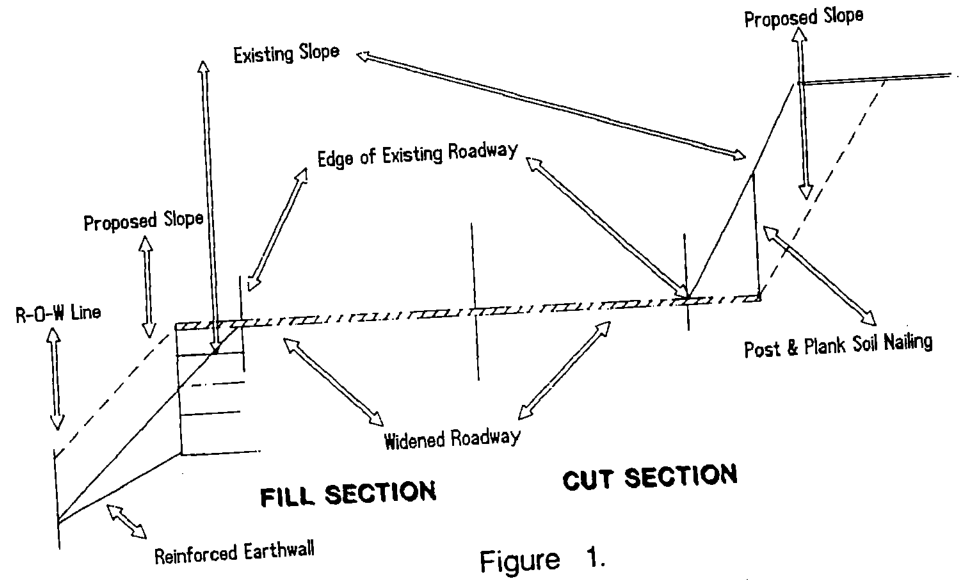


Figure 1.

means of conventional slope-building techniques. In areas of limited right-of-way, the Pennsylvania Turnpike has made extensive use of reinforced earth walls (see Figure 1). In addition, Also, geogrid products have been used to build up fill slopes. The geogrid increases the soil's tensile strength and allows for a steeper slope than those provided by conventional construction.

LEHIGH TUNNEL No. 2

Background

The only remaining section on the Pennsylvania Turnpike with only two lanes available to traffic is at the Lehigh Tunnel approximately 13 miles north of Allentown. The two-lane Lehigh Tunnel No. 1 was constructed and opened to traffic in 1957. Estimates of traffic volumes made during planning of the Turnpike's Northeast Extension warranted only a single two-lane tunnel.

In the early 1970s, bids were solicited for the construction of a new Lehigh Tunnel. However, the Lehigh Tunnel No. 2 project was put on hold for 16 years because the original bids were far above estimates. In the mid-1980s, though, increased traffic volumes and weekend backlogs at the tunnel warranted a new search for possible alternatives. These alternatives were a bypass around the mountain or Lehigh Tunnel No. 2. The bypass was ruled out because it would have to pass through an environmentally sensitive area of the Appalachian Trail. In addition, in Act 61 the Pennsylvania Legislature mandated the construction of a tunnel as part of the Turnpike Expansion.

The original Lehigh Tunnel No. 2 bid documents were revised and updated. The bids included construction by means of a conventional tunneling method or the New Austrian Tunneling Method (NATM). The conventional method, similar to that used for Tunnel No. 1, included a cast-in-place reinforced concrete arch. The NATM, used predominantly in Europe, takes advantage of the inherent strength of rock masses.[3]

Geological Cross Section
Along Lehigh Tunnel

(NOT TO SCALE)

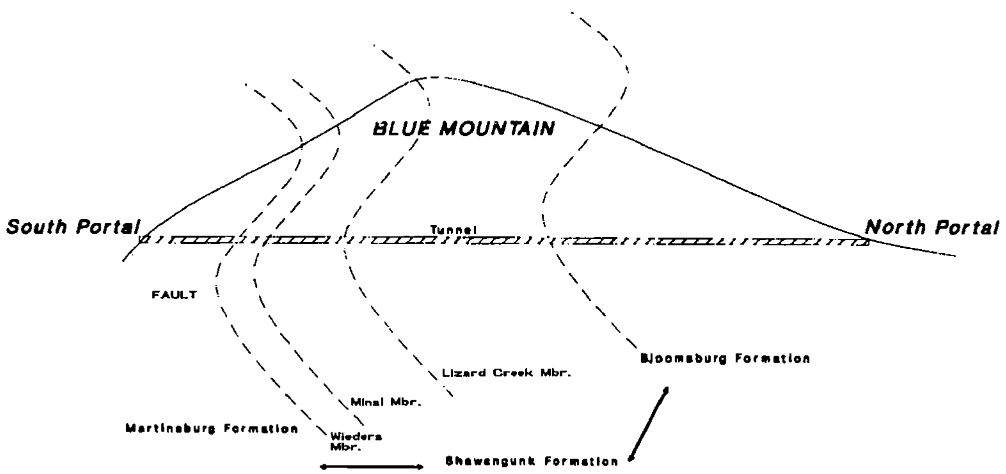


Figure 2.

<u>Geology</u>

Lehigh Tunnel No. 2 is set inside the Blue Mountain in Pennsylvania. The Appalachian Trail runs directly across the tunnel, at the top of the mountain. Figure 2 plots the mountain's stratigraphy.

The rock strata in this area are sedimentary and dip from 30° to 40° towards the south. Rock types include shale, siltstone, sandstone, conglomerate, and quartzite. The rock formations are from the Late Ordovician to the Early Silurian Periods. The Lehigh Gap is an exposed rock surface eroded and weathered by the Lehigh River after millions of years.

A rock classification system was developed to distinguish the various construction sequences necessary to support the initial tunnel lining. The following classes were used:

Rock Class 1 *- Sound rock, moderately jointed, stable during construction. (Ex. conglomerate, sandstone, consolidated shale)*

Rock Class 2 *- Moderately to closely jointed rock, roof conditions moderate, general rock mass with limited stand-up time (1 day) (Ex. consolidated sandstone, shale, siltstone)*

Rock Class 3 *- Fractured rock with closely spaced joints and shear zones, roof conditions brittle, rock mass unstable, stand-up time of several hours. (Ex. fractured sandstone, shale, siltstone)*

Rock Class 4 *- Weathered rock, intensely sheared fault zones, poor roof conditions, rock mass unstable, short stand-up time, support of excavation face required. (Ex. highly fractured sandstone, shale, siltstone.)*

Rock Class 5 *- Decomposed rock, roof conditions very poor, roof needs pre-support.*

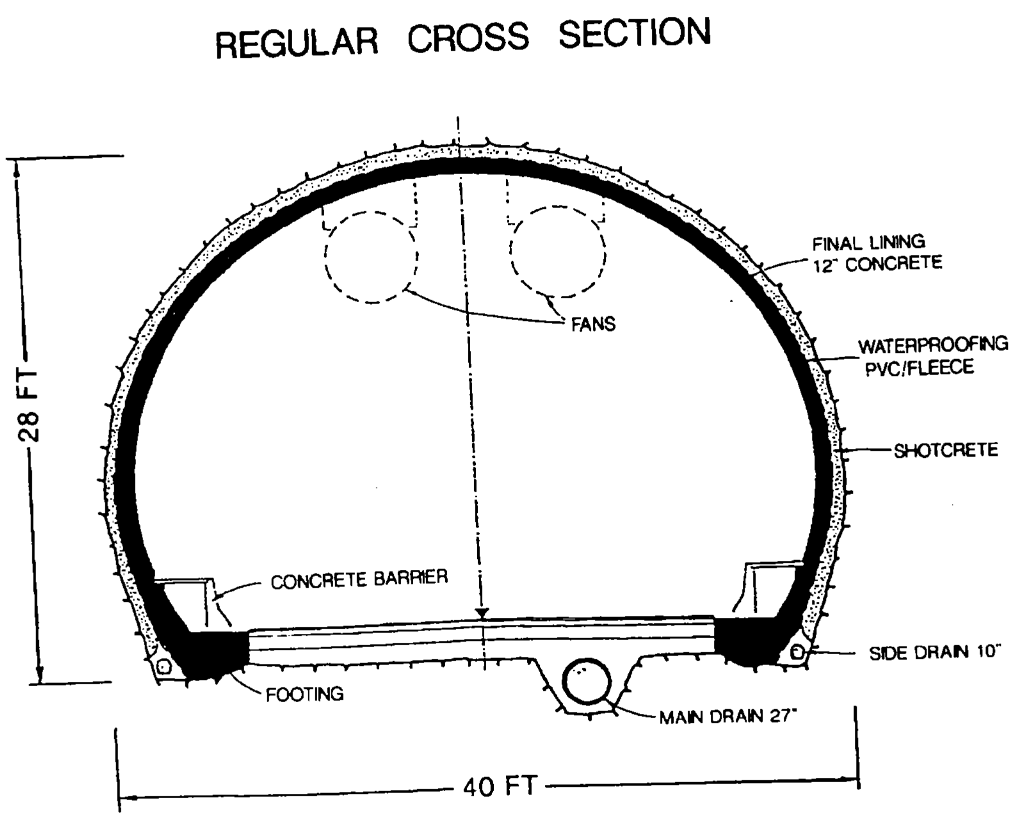


Figure 3.

<u>Construction</u>

During the bid phase of the project, bids were accepted for either the conventional tunneling method or the NATM. The lowest bid for the tunnel construction contract, in excess of $37 million, was for use of the NATM. This project would be first application of NATM to build a vehicular tunnel in the United States. The only other NATM projects in the U.S. at the time were the Mt. Lebanon Railroad Tunnel, and a section of tunnel for the Washington, D.C. Metro subway.

Figure 3 is the typical cross section for Lehigh Tunnel No. 2. As depicted in the diagram, the tunnel includes an initial lining, a waterproofing membrane, and a final concrete lining.

NATM is a very flexible construction method. It adapts itself to varying rock conditions underground. After excavation, an initial application of shotcrete stabilizes the newly created load-bearing arch. Depending on rock class, treatments of the initial lining include rock bolts and additional shotcreting with mesh and lattice girders. Deformations are constantly monitored to ensure that supports in place are sufficient to maintain the inner strength of the arch. The treatments of the initial lining are listed by rock class in Table I.

Tunnel construction has been progressing rapidly. The amounts of rock encountered, by rock class are shown in Table II.

At present, all excavation inside the tunnel is complete. The final waterproof lining is near completion. It is expected that all concrete final lining pours will be completed by mid-April 1991. Items that remain to be completed include roadway (inside tunnel and approaches), coating of concrete lining, lighting, fan installation, and portal construction. It is anticipated that Lehigh Tunnel No. 2 will be opened to traffic in November 1991.[4]

Table I

INITIAL TUNNEL LINING TREATMENTS				
Rock Class	**Advance Rate Per Excavation Round**	**Shotcrete**	**Roof Bolts**	**Lattice Girders**
Class 1	12' Top Heading 24' Bench	During Excavation	Occasional (10' to 15')	No
Class 2	7'-9'Top Heading 14'-18' Bench	Exposed Rock Surfaces to a Minimum 2"	Occasional (10' to 15')	No
* Class 3	6'-7' Top Heading 12'-14' Bench	Minimum 6" with wire mesh	7 to 8 per Round (10' to 15')	1 per Round
* Class 4	4'-5' Top Heading 8'-10' Bench	Continuous Reinforced with mesh Minimum 8"	9 to 10 per Round (10' to 15')	1 per Round
* Class 5	4' Top Heading 8' Bench	Continuous Reinforced with 2 mesh layers Minimum 12'	14 per Round (10' to 20')	1 per Round

**Possible use of rebar spilling as advance support*

Table II

Rock Class	Footage
Class 1	2769'
Class 2	935'
Class 3	113'
Class 4	298'
Class 5	127'

ROCK CUTS

Background

Many portions of the Pennsylvania Turnpike were constructed in the 1930s through the 1950s. Highways cut during that period have seen many weathering cycles, which have taken a serious toll. In addition, the construction standards of that era are considered substandard by today's guidelines. As a first step to alleviate some of these problems and to restore integrity to some of the aging slopes, the Turnpike Commission authorized its consulting engineer to study all rock cuts, systemwide, with the exception of newly reconstructed areas.

Design

The rock cuts were evaluated on the basis of geometry, geology, physical characteristics, and drainage. In addition maintenance personnel were interviewed about the frequency of falls. The study of the system rated rock cuts on a scale from 1 to 5. Category 1 rock cuts were deemed to be safe, with no immediate action required. At the opposite end of the scale, the Category 5 rock cuts were deemed to be in need of repair. The study identified six Category 5 rock cuts that were in need of some type of repair.

Based on this recommendation, all six of the Category 5 rock cuts were studied for possible repair alternatives. The feasibility studies looked at a minimum of three alternatives for final repair. The alternatives had to take into account the type of construction, right-of-way, maintenance and protection of traffic, and cost. The feasibility studies were used as a basis for final design.

A consultant designer was chosen to take the Category 5 rock cuts to final design. The preliminary final design was a cost comparsion of two methods. The actual final design was based on the best alternative. The final design configuration of the slope was based on a comparison of the two proposed alternatives. Type of construction, ease of construction, right-of-way requirements, maintenance and protection of traffic, and costs were considered in the comparison.

Construction

To date, five of the six Category 5 rock cut slopes have been repaired. The construction included such processes as shotcreting a weathered coal seam, drilling and blasting, installation of rock fall fence, and huge amounts of rock excavation. Bids ranged from $300,000 for a small cut to approximately $2,000,000 for a large cut.

All construction work had to be completed while maintaining smooth traffic flows. The only disruption to patrons were daily traffic stoppages that did not exceed 15 minutes. The traffic stoppages were necessary to protect the motoring public as the rock to be excavated was blasted and fragmented. All blasts were monitored by seismograph to ensure that all blasts met state, local and federal guidelines.[5]

CONCLUSION

This material is presented as an informative illustration of construction practices being used on the Pennsylvania Turnpike and the complexities that they involve. It is hoped that the information presented here will benefit others in the pursuit of safer, less expensive, and superior construction practices for the highway industry.

ACKNOWLEDGEMENTS

The author would like to thank Mr. William Brunner of the Dr. Sauer Corporation for providing a wealth of information on the construction of Lehigh Tunnel No. 2. The author would also like to thank Connie Kreischer for her assistance with graphics and editing.

ENDNOTES

1. Dan Cupper, The Pennsylvania Turnpike A History, (Lebanon, PA: Applied Arts Publishers, 1990)

2. Dan Cupper, "The Pennsylvania Turnpike Celebrates Its 50th Anniversary," Pennsylvania Magazine, 10, No. 3 (February 1990),
pp. 6-14.

3. Cont. No. 02-051-C980-C, The Construction of Lehigh Tunnel No. 2, Roadway and Approaches, by The Pennsylvania Turnpike Commission, August, 1988.

4. Geotechnical Report for Lehigh Tunnel No. 2; NATM Alternate Design; Geotechnical Basis of Design Tunnel, by Dr. Gerhard Sauer Corp., McCormick Taylor & Associates, Inc. a Joint Venture for The Pennsylvania Turnpike Commission, August, 1988.

5. In Depth Inspection Rock Cut Slopes, **Milepost A-26.9 to A-27.2, Northbound; Milepost A-85.5 to A-86.1 Northbound and Southbound; Milepost 83.05 to 83.5 Westbound; Milepost 115.9 to 116.1 Eastbound and Westbound.** by Michael Baker, Jr., Inc. for the Pennsylvania Turnpike Commission, Revised February 1988.

Seismic Refraction Technique Applied to Highway Design in a Strip Mine Area of Southwestern Pennsylvania

Douglas Rudenko, Vibra-Tech Engineers, Inc.
Walter M. Lawrence, Geomechanics, Inc.
Hans D. Ackermann, Vibra-Tech Engineers, Inc.

NTRODUCTION

modification of the standard seismic refraction echnique, normally employed to measure bedrock depth nd quality, was used to determine highwall locations and epths to in-place rock along S.R. 6119, Section A08--a ortion of the Mon-Fayette Expressway to be constructed n Georges Township, Fayette County, in southwestern 'ennsylvania. Figure 1 shows the exact location of the ite.

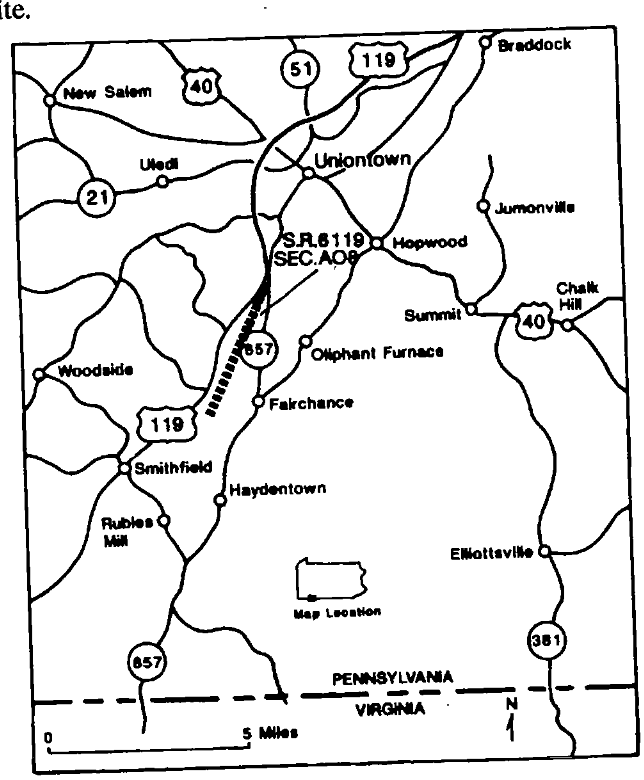


Figure 1: Site location map.

A prior test boring and engineering analysis program identifed the general extent of past coal removal and recommended methods of treatment. The results of the seismic refraction study were used during final design to more accurately identify the various treatments necessary to ensure safe roadway behavior through areas previously strip-mined.

The planned construction project consists of a 4.3-mile section of a divided four-lane, limited-access expressway (GeoMechanics, 1990). The purpose of the project is to enhance economic growth opportunity in Georges Township, Fayette County by providing the needed stimulus for development of the Fairchance Industrial Park. S.R. 6119, Section A08 will ultimately become a part of the Mon-Fayette Expressway, which stretches from Pittsburgh to West Virginia. This project has been designated as one of seven pilot projects in the United States which permits federal participation in funding for the construction of a toll facility that will be operated by the Pennsylvania Turnpike Commission.

SITE GEOLOGY

The alignment of S.R. 6119, Section A08 lies within the Appalachian Plateau physiographic province in the western part of Fayette County, near Uniontown, Pennsylvania. Topographically, this area consists of rolling uplands, dissected by many steep-sided stream valleys. The roadway alignment lies on the eastern limb of the Uniontown Syncline. The general trend of the fold axis is NE-SW. The strata dips approximately 0.65° in a N30W direction.

Stratigraphically, bedrock units in the study area progress from the upper section of the Conemaugh Group at the southern end of the alignment, through units of the Monongahela Group, to the lower section of the Washington (Dunkard) Group at the northern end. The Conemaugh Group and the Monongahela Group are of Pennsylvanian age, and the Washington (Dunkard) Group is of Permian age. The Conemaugh Group is bounded at its base by the top of the Upper Freeport coal seam and by the bottom of the Pittsburgh coal seam at its top. The Group consists of cyclic sequences of shales, sandstones, thin limestones, and thin coal seams. The Monongahela Group extends from the bottom of the Pittsburgh coal seam at its base upward to the bottom of the Waynesburg coal seam at its top. This Group is the dominant rock sequence in the project area and shows a wide variation in lithology from place to place. It consists of shales, limestones, some coarse sandstones, and several mineable coal beds. The Washington (Dunkard) Group includes rocks above the bottom of the Waynesburg coal seam. Only about 160 feet of basal section remains uneroded in the hilltops of the S.R. 6119, Section A08 corridor. These beds consist of drab sandstones, sandy shales, shales, and coals.

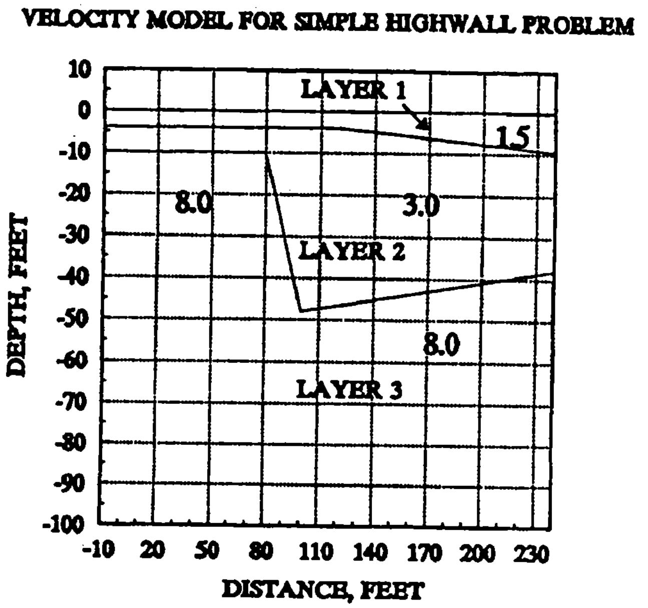


Figure 2A: Hypothetical velocity model for a simple highwall problem. Velocities in kilofeet/second.

MINING HISTORY

The Monongahela Group coal seams have been exploited throughout the entire corridor of S.R. 6119, Section A08. The Pittsburgh coal seam is the principal coal resource of Fayette County, followed in importance by the Sewickley and Waynesburg coals. The Redstone and Uniontown coals are of lesser importance.

The Pittsburgh coal outcrops near the southern end of the corridor and is persistently present north of this point. Deep-mining of the coal under the corridor began by means of room-and-pillar techniques in the early 1890's and continued through the early 1940's. After that time, retreat-mining of the pillars was performed where conditions permitted. Detailed mapping indicates that approximately 80% of the coal has been retreat-mined, and very little support is left in the mined-out areas. The Pittsburgh coal has also been strip-mined along its outcrop. The seismic cross section we examine in this paper runs through the outcrop of the Pittsburgh coal that has been both deep-mined and strip-mined.

DATA ACQUISITION

A total of 7000 linear feet of seismic profiling was completed for this project. Twenty-eight seismic traverse lines 250 feet long were run. The lines were parallel and perpendicular to roadway centerline; therefore they were roughly perpendicular to the assumed axis of the buried highwall.

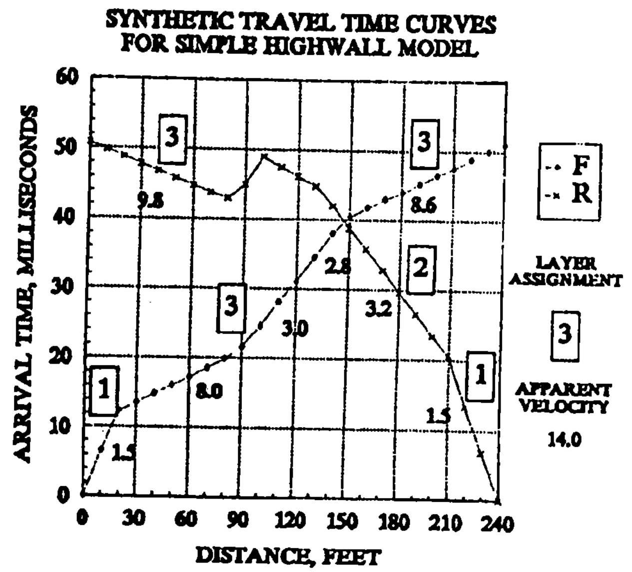
A 24-channel, Scintrex Model S-2 Echo seismograph was used to record the field data. On each of the traverse lines, 24 geophones, spaced 10 feet apart, were placed along the ground surface. In order to acquire sufficient data to accurately determine the presence, location, depth, and size of a highwall, each seismic spread was recorded from several shotpoint locations: two within the spread, two at each end, and two offset approximately 115 feet from each end. Data from various shotpoints were combined to provide internally consistent travel-time curves from several depths beneath the surface, to a total depth of approximately 100 feet (Ackermann, et al., 1986).

SIMPLIFIED HIGHWALL MODEL

In this paper we discuss the different seismic models that could be obtained by means of a less than optimum number of shot points for a given spread located over a highwall, and how the derived velocity models may improve with the addition of extra shotpoints. Figure 2a shows a simplified seismic model of a buried highwall. The 1500 ft/sec layer represents unconsolidated near-surface material. The back-filled strip-mine spoils are represented by the 3000 ft/sec velocity, and bedrock is represented by the 8000 ft/sec velocity. The top of the highwall is located 80 feet from the left axis and the toe at 100 feet.

Two shotpoint locations at 0 feet and 240 feet were assumed to construct the synthetic travel time curve shown in Figure 2b.

The synthetic travel-time curve is drawn to the same



ᴾigure 2b: Synthetic travel-time curves calculated from Figure ²a. Shotpoints are located at 0 and 240 feet. The approximate ocation of the top of the highwall occurs on both curves at the change in slope near 80 feet. The approximate location of the oe of the highwall can only be seen on the reverse shotpoint near 100 feet. Travel time curves for two shotpoints (F and R) ocated 10 feet off either end of the spread. The layer assignments are labeled. The approximate location of the top of the highwall can be seen on both curves at the change in slope near 170 feet.

norizontal scale as the model for comparison. The travel ime curve shows that we can expect very abrupt changes n slope in the immediate vicinity of the highwall. The forward shotpoint at 0 feet shows only the top of the nighwall because the toe is located in a blind zone due to he abrupt geometry of the highwall. The reverse shotpoint located at 240 feet, however, indicates both the oe and the top of the highwall.

CASE 1 – TWO SHOTS OFF END

With the synthetic model in mind, actual field data will now be examined. Case 1 consists of two shotpoints recorded ten feet off each end of the spread. Figure 3a shows the real travel time curve, having essentially the same geometry and shot configuration as the synthetic model from which any number of velocity models could be calculated.

One realistic interpretation that assumes that layering on the left side of the model is parallel to the ground surface. is shown in Figure 3b.

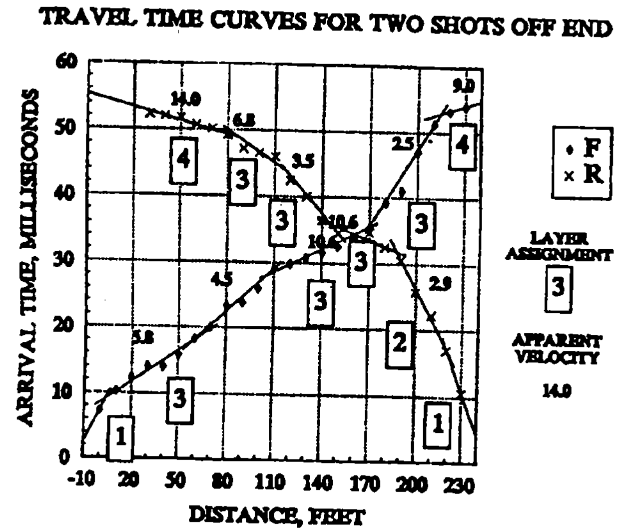


Figure 3a: Travel-time curves for two shotpoints (F and R) located 10 feet off either end of the spread. The approximate location of the top of the highwall can be seen at the change in slope near 170 feet on both the F and the R curve. The approximate location of the toe of the highwall is somewhere to the left of the change in slope near 210 feet on curve R.

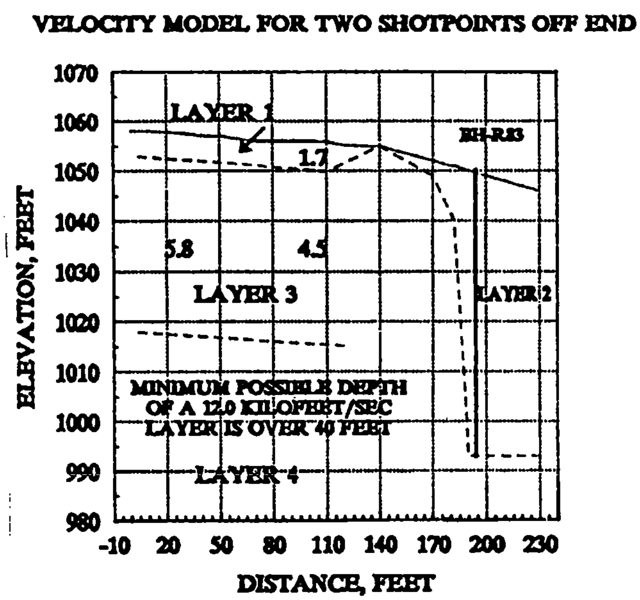


Figure 3b: Velocity model determined from Figure 3a. Layer assignments correspond to those shown in Figure 3a. Velocities are given in kilofeet/second. Borehole R-83 showed top of rock to be at 56 feet. Vertical exaggeration is 3:1.

In Figure 3a the layer assignments are labeled with large print integer values, and the apparent velocity of each layer is given with smaller print values. The velocity model generated in Figure 3b suggests the following:

1. The top of the highwall occurs at approximately 170 feet along the distance axis.

2. The toe of the highwall is somewhere to the left of 210 feet.

3. A thin layer of 1.7 kilofeet/second (kft/s) material is underlain by a layer where velocity changes from 5.8 to 4.5 kft/s.

4. A high velocity material (about 12.0 kft/s) occurs over 40 feet deep.

The location of the toe of the highwall could not be determined from the seismic results for this case. Instead, borehole data to the right of our assumed location for the top of the highwall were used to locate the toe at a depth of approximately 56 feet.

TRAVEL TIME CURVE FOR FOUR SHOTS OFF END

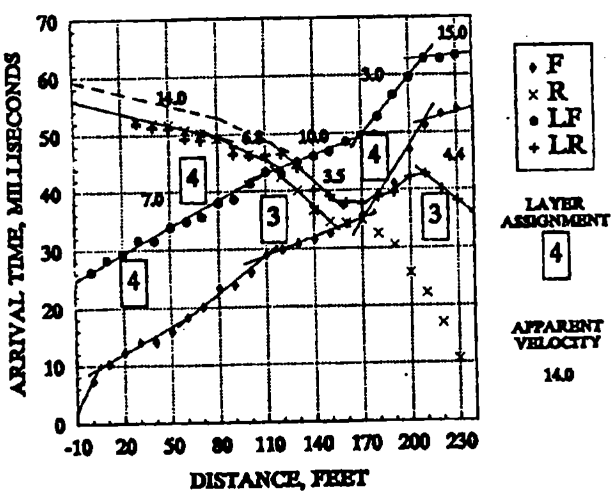


Figure 4a: Travel-time curves for two additional shotpoints (LF and LR) located 115 feet off either end of the spread. The approximate location of the top of the highwall can be verified at the change in slope near 170 feet on both the LF and LR curves. The approximate location of the toe of the highwall can be seen only on the LR curve at the change in slope near 210 feet.

CASE 2 – FOUR SHOTS OFF END

Case 2 consists of four shotpoints. Information from two additional shotpoints (LF and LR) located 115 feet off either end has been added to compute a new velocity model. Figure 4a shows the travel-time curves, with the layer assignments and apparent velocities marked for the two additional shotpoints.

VELOCITY MODEL FOR FOUR SHOTPOINTS OFF END

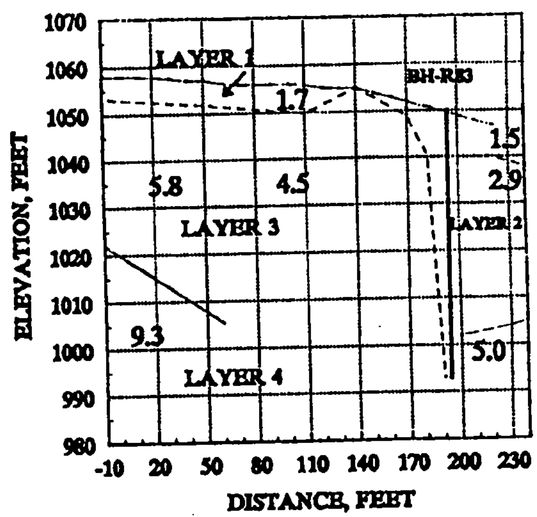


Figure 4b: Velocity model determined from Figure 4a. Layer assignments correspond to those shown in Figure 4a. Velocities are given in kilofeet/second. Borehole R-83 showed top of rock to be at 56 feet. Vertical exaggeration is 3:1.

Offset shots are usually added to the shooting geometry to improve data from deeper layers.
Figure 4b shows the velocity model calculated using the information collected from the additional shotpoints.

This velocity model establishes a few more points about the subsurface.

1. Shotpoint LF reconfirmed the location of the top of the highwall and also enabled us to calculate the velocity, depth, and dip of the high velocity (9.3 kft/s) layer to the left of the highwall (Dobrin, 1976) (Palmer, 1980)

2. Shotpoint LR verified the approximate location of the toe of the highwall and provided information about rock velocities and depth to the right of the highwall.

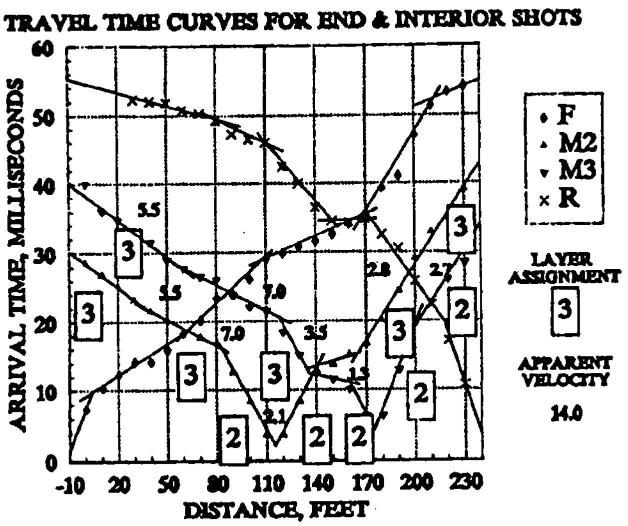


Figure 5A: Travel-time curves for additional shotpoints (M2 and M3) within the spread.

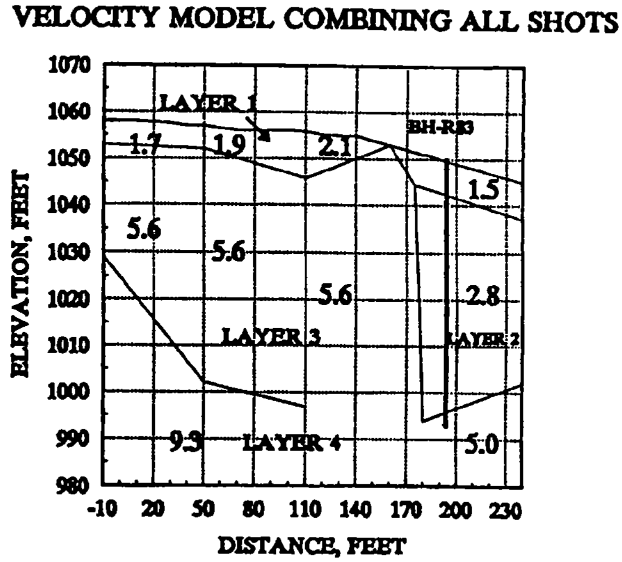


Figure 5b: Velocity model determined from Figure 5a. Layer assignments correspond to those shown in Figure 5a. Velocities are given in kilofeet/second. Borehole R-83 showed top of rock to be at 56 feet. Vertical exaggeration is 3:1.

CASE 3 -- ALL SHOTPOINTS

Case 3 added two more shotpoints (M2 and M3) within the spread used to obtain the final velocity model. Figure 5a shows the travel-time curves for the additional interior shotpoints.

For clarity, only the end shotpoints (F and R) are displayed with the interior shotpoints (M2 and M3). Shotpoints within the spread generally provide additional subsurface information for the same depth as the end shotpoints, or for shallower depths. Figure 5b shows the final interpreted velocity model. Depths calculated in the seismic model are in good agreement with the borehole data. The interior shotpoints provide the following information:

1. They fix the velocities of layer 2 (2.8 kft/s) to the right of the highwall and layer 3 (5.6 kft/s) to the left of the highwall.

2. From these velocities we are able to calculate the depth and dip of layer 4 and thereby determine location of the toe of the highwall without the aid of borehole data.

CONCLUSIONS

A seismic refraction spread was recorded from six shotpoints; two within the spread, two at each end, and two more offset 115 feet from each end. Three cases were examined. The first case utilized only the two end shotpoints. The second examined additional data provided by the two offset shotpoints. The third case included the interior shotpoints. Travel-time curves were produced and analyzed and separate velocity models constructed for each case.

The first case provided minimal information about the subsurface. We were able to distinguish the location of the top of the highwall and estimate the number of layers and their velocities. Depth and velocity information on these layers was questionable.

Case 2 further substantiated the location of the top of the highwall and provided information on a deeper layer to the left of the highwall. Information was also obtained about the approximate location of the toe of the highwall and about the velocity and depth to rock to the right of the highwall.

The third case further refined Case 2 by pinning down the velocities of layers 2 and 3, to the right and left of the highwall respectively, thus enabling us to make a more precise calculation of the depth and dip angle of layer 4. Furthermore, these velocity values were critical for the determination of the base of the highwall.

ACKNOWLEDGEMENTS

The writers wish to express their appreciation to the Pennsylvania Department of Transportation, and in particular Engineering District 12-0, for recognizing the usefulness of seismic refraction techniques for identification of buried strip-mine highwalls and for providing the necessary funding to complete the seismic surveys, and to Sucevic, Piccolomini and Kuchar Engineering, Inc., the prime consultant for the design of S.R. 6119, Section A08, which coordinated the field work and provided the ground controls needed to complete the work in a timely manner.

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EVALUATION OF ACID LEACHATE POTENTIAL IN HIGHWAY CONSTRUCTION

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ABSTRACT

Acid leachate, runoff with low pH and high sulfate concentration, has been a concern for many years in mining operations and more recently in highway construction within the coal mining regions. Increased concern about the quality of surface and groundwater resources has resulted in a greater knowledge of factors that affect these resources. However, unlike toxic or hazardous materials, there are no regulatory standards for concentrations of acid leachate. Therefore, government agencies and private consultants must use chemical, geologic, and engineering analyses to evaluate the potential for detrimental impact on the environment.

The chemical process that produces acidity is reviewed and the impact of acid leachate on the environment and construction materials is discussed. Methods to identify acid leachate potential by means of geological studies and laboratory tests are described. Once the potential is quantified, methods for controlling production of acid leachate or limiting impact through engineering design and construction practices can be developed. Some typical procedures are presented.

INTRODUCTION

Increasing public environmental awareness and government regulations have resulted in strict guidelines that must be followed during land development. One of the most visible and disruptive types of development is highway construction or renovation. These projects may involve many miles of new roads, sometimes with significant grade changes, crossing or filling of inland wetlands, or alterations to local hydrology.

With increased concern about the quality of surface and groundwater comes greater knowledge of factors that affect these resources. One factor that may influence water resources is the chemical effect, specifically acidity, of the rock exposed or used during construction.

Controlling acid leachate (low pH, high sulfate runoff) has been a concern for many years in coal mining. This concern has resulted in adoption of abandoned mine reclamation programs in many states. It has been shared by highway departments throughout the Appalachian region since many of the same rock formations and coal seams were encountered in highway construction.

The concern about acid leachate has begun to spread to the northeast states where metamorphic rocks dominate. The acid leachate potential within non-sedimentary rocks is significantly less than within the sedimentary rocks of the coal mining regions. However, specific rock types are very likely to have a serious effect on the environment.

Unlike toxic or hazardous materials, concentrations of acid leachate are not subject to regulatory standards. Therefore, it is the responsibility of consultants and highway departments to evaluate the acid leachate potential, its impact on the environment, and to identify remedial measures. To do so, they must characterize the rock types they expect to encounter during construction, identify the acid leachate potential through mineral classification and laboratory tests, and develop engineering measures and construction procedures that will control the effect of the rocks that could produce acid.

GENERATION OF ACID LEACHATE

Acid leachate is produced when iron sulfide minerals are exposed to air and water and oxidize to form soluble hydrous iron sulfates. When water flows over the weathered rock surface, the compounds dissolve and hydrolyze to form acidic, high sulfate, high iron leachate.

The Chemical Process

The commonly accepted chemical reactions that cause oxidation of iron sulfide (FeS_2) and production of acid (H^+) are (Barnes & Romberger, 1968, and Baker, 1975) as follows:

(1) $2FeS_{2(s)} + 7O_2 + 2H_2O = 2Fe^{2+} + 4SO_4^{2-} + 4H^+$

(2) $Fe^{2+} + 1/4O_2 + H^+ = Fe^{3+} + 1/2H_2O$

(3) $Fe^{3+} + 3H_2O = Fe(OH)_{3(s)} + 3H^+$

(4) $FeS_{2(s)} + 14Fe^{3+} + 8H_2O = 15Fe^{2+} + 2SO_4^{2-} + 16H^+$

Eq. 1 shows 1 mole of FeS_2 producing 2 moles of acid. Eqs. 2 and 3 show Fe^{2+} oxidizing into Fe^{3+} and producing an additional 3 moles of acid. It has also been shown (Baker, 1975) FeS_2 can oxidize in the presence of excess Fe^{3+} in solution with water and further hydrolyze to produce 16 moles of acid (Eq. 4).

The process is a chain reaction. The ferric ion produced in Eq. 2 is used in Eq. 4 to oxidize pyrite. There is a net reduction to ferrous ion, which is then reoxidized in Eq. 2 to ferric ion to become available again for Eq. 4. The only reactant consumed is pyrite, and the only product is sulfuric acid.

This process often occurs slowly, but it can be accelerated by iron-oxidizing bacterium such as Thiobacillus thiooxidans (a sulfur-oxidizing bacteria), Thiobaccillus ferrooxidans (a bacterium that oxidizes Fe^{2+} to Fe^{3+}), and Ferrobaccillus ferrooxidans (another sulfur-oxidizing bacteria) (EPA, 1971a cited in Caruccio and Geidel, 1978). The sulfur-oxidizing bacteria thrive in aqueous environments with pH values between 2.8 and 3.2.

In non-geochemical terms, the production of acidity is a three-stage process. In the first stage, the iron sulfide mineral, often pyrite, is oxidized by exposure to air or the sulfur-oxidizing bacteria. Initially, each rainfall slows the reaction by washing alkaline material (if available) into the acid-producing environment. However, if alkaline availability is limited or if acidity exceeds alkalinity, the reaction progresses into stage two and it becomes difficult to return to stage one.

In the second stage, the environment becomes acid-rich. The occurrence and distribution of the bacteria increase, the pH in the environment drops to the critical 2.8 to 3.2 range, and stage three is developed.

As the pH falls, the solubility of the iron increases and the effect of alkaline dilution decreases. The result is a dramatic increase in acidity and iron concentrations. Intermittent flushings of the environment at this stage by rainfall or groundwater result in highly acidified runoff. At this point, the only way to slow the process is to kill the bacteria.

Acidity

Sulfur occurs most commonly as organic sulfur, sulfate sulfur, and sulfide sulfur. The combination of these is called total sulfur. In certain geologic settings, total sulfur is considered a reasonable indicator of acidity potential.

Organic sulfur, found in coal, is considered organically bound within the coal and therefore not chemically reactive. Typically, organic sulfur is not present in metamorphic rock.

Sulfate sulfur is a by-product of weathering and therefore typically not present in fresh samples. If present, sulfate sulfur poses little hazard because it is relatively insoluble and may constitute only a small percentage of the total sulfur.

Pyritic sulfur (as iron disulfide FeS_2) is the sulfur contained in the sulfide phase. Pyrite and pyrrohotite are the dominant sulfides, although others, such as copper, zinc, and lead, may be present. Sulfide sulfur is considered the only hazardous form of sulfur.

Alkalinity

The rate at which acid is produced is also a function of the calcium carbonate available to neutralize it. As indicated above, sufficient alkalinity stops or delays progress to stage two, thereby preventing production of an acidic environment. Alkalinity may be available in calcareous rocks or overburden soils.

pH

Calcareous minerals can also provide neutralization potential through their effect on groundwater. In the bsence of calcareous material, the pH of natural groundwater could be less than 5.5. Iron-oxydizing bacteria can survive in this mildly acidic environment.

Temple and Koehler (1954) indicate the pH of water is also important in establishing the stability of certain types of pyrite. Tests indicated calcium and magnesium carbonate were present in stable pyrite nodules, but absent in reactive ones. After the protective calcium carbonate was removed by washing in an acid solution, the inert sample became reactive. Hence, once acidic waters are formed, the oxidation process occurs and the acid-generating chain reaction begins.

EFFECTS OF ACID LEACHATE

Once the highly acidified leachate enters surface or groundwater systems, it affects whatever it touches. If the volume of acidified water is significant, contamination begins. The chemical and physical characteristics of the contamination may persist long after the acid producing condition has been neutralized, because in-place pollutants continue causing problems until the water body cleans itself.

Chemical characteristics of water contaminated with acid leachate are low pH, high iron and sulfate concentrations, increased hardness, and significant amounts of aluminum, calcium, and manganese. Dissolved oxygen is depleted by the chemical combination of dissolved salts from the acid leachate and the oxygen in the water. The polluted water often turns reddish yellow; it has increased turbidity, suspended and dissolved solids, and precipitation of ferric hydroxide and ferric oxide.

These chemical and physical characteristics can have drastic effects on aquatic life (Allen, et al., 1978). Most aquatic fauna is destroyed below pH 4.5; between 4.5 and 5.2, fish behave abnormally and are deformed; at pH less than 6.6, egg production and egg hatchability is reduced.

Plant life may also be threatened. pH values below 3.5 will kill most species. Water with low pH (2-4.5) is capable of supporting only acid-tolerant molds and algae.

Acid water is unacceptable for recreational use, irrigation, and drinking by livestock or wildlife.

In addition to damaging the environment, acid-contaminated water corrodes to construction materials such as concrete and steel. Foundation systems, culverts, and drain pipes may be effected. The result may increase water treatment costs for municipal and industrial uses or even necessitate restrictions on the use of certain construction materials. For example, high sulfate concentration in soil and water may require use of Type V cement or a combination of Type V cement and pozzolan. Low pH may also require protective measures for concrete.

A corrosive environment may occur where zones of highly acidic soil are adjacent or mixed with less aggressive soils. When pipes pass through soils with varying soil resistivities, galvanic cells can be created that then produce a current, and corrosion begins. Soils with low pH generally have low resistivity. As pH decreases, the corrosion rate of steel increases (T. Steffens, 1990).

EVALUATION OF ACID LEACHATE POTENTIAL

It is impractical to test all rock within the proposed construction limits to identify acid leachate potential. Reviewing geological maps and performing geological reconnaissance for minerals and geologic features that enhance acid production will rule out certain formations and highlight others. Test screening selected samples within the highlighted regions will provide quantitative data to determine if conditions are likely to produce acid. If all test screening results are far from "threshold" values, further testing may not be required. If results are near the threshold, a testing program of systematic sampling and a range of laboratory procedures likely will be required.

Rock Types Favorable for Acid Production

The principal acid-producing mineral is iron sulfide, FeS_2. The most common form of iron sulfide is pyrite; less common forms are pyrrhotite and marcasite. Rock types with relatively high concentrations of these minerals will likely produce acid.

These iron sulfide minerals exist in various concentrations in many metamorphic schists and occasionally in igneous intrusions. Some sedimentary rock groups within the northeast are also know for high sulfide concentrations.

Within New England, metamorphic schist and gneiss are the predominant rock types. The parent rocks are varied and often poorly understood. In New York State, extensive areas of sedimentary shale, sandstone, and limestone exist. With the exception of the "black shales," the majority of the sedimentary rock units do not appear to have significant quantities of sulfide minerals and therefore are not considered a serious threat.

Metamorphic units of the northeast frequently occur as elongated bands extending northeast-southwest. Consequently, problem units may be quickly traversed in the perpendicular direction, or followed for many miles in a parallel direction.

During geological reconnaissance, a simple but useful way to classify rocks is by their overall color. Dark colors generally indicate mafic (iron-rich) rocks. Light colored rocks are normally quartz-rich and iron-poor.

The predominant minerals within iron-rich metamorphic rocks are biotite, amphiboles, and pyroxenes. As a result, these rocks tend to be fine-grained and easily weathered. Prominent preferred cleavage planes will produce weak rock slopes and fractures through which water and oxygen can move and react.

Iron-rich rocks will generally include iron sulfides if sulfur was available within the parent rock. During metamorphism, the iron and sulfur combine to form iron sulfide (most commonly pyrite) as an accessory mineral. Because of the physical properties described, chemical and physical weathering will tend to expose fresh surfaces to further oxidation. Recently exposed surfaces of iron sulfide-bearing formations may have a characteristic rusty brown to yellow brown coating. As a rule, if pyrite can be readily identified in representative hand specimens and is believed to be widespread, then a strong acid leachate potential exists.

Metamorphic rocks generally have one or more well developed joint sets, especially parallel to the foliation or layering within a schist or gneiss. These fractures should be evaluated for their potential to enhance groundwater movement towards a proposed cut. Such fractures may be opened further by blasting.

Alkaline-producing materials may provide sufficient buffering or neutralizing capabilities to avoid acid production. Alkaline materials may be present in calcareous rock or in the overburden, especially glacial till. These alkaline soils will tend to neutralize acidity by raising the pH of percolating groundwater prior to contact with the suspect rock. County soil surveys provide information on pH of soils.

Laboratory Testing

Geological maps, and often direct observation of exposed rock or rock core samples, do not provide sufficient detail or precision to confirm the potential for acid leachate. Therefore, laboratory tests should be performed.

Traditional methods for identifying rock mineralogy, such as thin section analysis and x-ray diffraction, are accurate to within about 2 percent. Iron sulfide concentrations of less than 2 percent can produce acid leachate.

Total sulfur analyses is a reproducible, relatively simple, and inexpensive test. This test detects all forms of sulfur, including the relatively non-reactive sulfate and organic sulfur. Testing for sulfidic sulfur alone has not proved reproducible. Therefore, the total sulfur test is considered by many states to be the standard screening test.

Highway departments in states within the Appalachian region that have encountered acid producing rock formations have used the data available from the mining industry to develop standards or threshold values for total sulfur. Current standards in Pennsylvania and North Carolina advocate the threshold value of 0.5 percent total sulfur (Morrison, 1989). This value was derived from laboratory tests for total sulfur vs. acidity using both short- and long- term methods. Tennessee uses a slightly lower threshold value, 0.3 percent total sulfur (Moore, 1989).

If total sulfur testing indicates near-threshold values, the neutralization potential (NP) of the material should be evaluated. This combined method is known as net acid-base account (Miller, 1988). The NP results are expressed in tons of calcium carbonate per 1000 tons of material. Total sulfur is converted to pyritic sulfur by applying an empirically derived constant to give results also expressed in tons of calcium carbonate per 1000 tons of material. The difference provides a Net Acid Producing Potential (NAPP). It has been observed (Caruccio, 1980) that the natural neutralizing capacity of many materials can strongly override even relatively high sulfide levels.

f the deficiency of calcium carbonate is greater than or :qual to 5 tons per 1000 tons of material, the material ;enerally is classified as potentially acid-producing. This tandard is being used by the Departments of Transportation of North Carolina and Tennessee (Moore, 1989 and Koch, 1989).

Another laboratory test for determining acid leachate potential is long-term leaching tests. These tests attempt to duplicate conditions that are anticipated in the field. Bulk samples are exposed to simulated weathering conditions and the resultant leachate is measured for pH.

Proponents of this test believe the maximum acidity predicted by the acid-base account may not accurately represent the actual acidity potential. The rate at which acid is produced in the field may exceed the rate of base production, thereby creating acid leachate that may not have been predicted by the acid-base account.

The primary problem with the leaching test is establishing an appropriate model to represent field conditions and interpreting the test results. In addition, the test is costly and takes many weeks to run. For these reasons, we believe leaching tests should be limited to critical conditions and the results used in conjunction with other tests.

Sampling

Since it still remains impractical to test screen all rock, a systematic sampling procedure should be followed with the realization that conditions will laterally and vertically. Samples must be closely enough spaced to identify possible localized sulfide occurrences that reflect original depositional variability and subsequent metamorphic remobilization. The neutralizing components of a particular unit may be relatively homogeneous, reflecting typical depositional modes for calcium carbonate precipitation (Caruccio, 1980).

Where distinctly different units are traversed, each should be sampled separately with attention paid to suspect units described earlier. If rocks display alternating layers too close to be effectively segregated, tests should be conducted on random samples of the overall rock mass.

ENGINEERING REMEDIATION

If acid-producing rocks exist at the site, several methods may be used to limit damage to the environment. Selection of the type and details of a remediation method depends on the magnitude of the problem and the site conditions. Identifying the level of acid leachate potential at the time of design, especially preliminary design, may allow revisions to the project that will help to minimize impact.

Alignment Changes

Changes in vertical or horizontal alignment of the proposed highway may lessen the potential for intersecting potentially acid-producing rocks. Horizontal changes may avoid a potentially acidic rock formation, or allow the grade modifications to have less impact. Where possible, placing a roadway on total embankment fill or in total cut is more desirable than a combination side hill cut and fill. Total fill is preferred because it will reduce the volume of acidic rock to be disposed of or covered, and likely it will have less impact on groundwater flow characteristics.

Storm Water Management

Surface runoff should be diverted from rock cuts and fills by paved drainage ditches or diversion berms at the top and toe of slopes. Water courses can be diverted, or channeled in pipes or conduits.

Several runoff neutralization procedures have been successful in reclamation of coal mine sites and can be implemented in highway construction. Detention ponds may be designed to collect and treat storm water runoff. The water quality is monitored, and neutralizing material (such as lime) added as necessary to balance the pH before the water is released. Surrounding storm water diversion is particularly important, for this approach, to limit the volume of water requiring treatment.

A second treatment procedure is neutralization of runoff with limestone-paved ditches. This procedure is suitable for mildly acidic runoff. Highly acidic runoff will cause ferric hydroxide precipitates to coat the limestone and thereby limit the system's effectiveness.

Biological treatment of acid leachate by either natural or man-made wetlands may be considered. The Tennessee Valley Authority has experimented with treatment of acid mine drainage by diverting it through man-made wetlands (B. Dawson, 1989). Positive results have been achieved at several coal mining sites in Tennessee, Alabama, and Kentucky.

Rock Fill

Lift thickness and compaction must be controlled to minimize fracturing of the rock, which would increase surface area exposed to oxidation and acid production.

Acidic rock should not be placed where it can come in contact with water. This is typically referred to as the "high and dry" approach.

Where the rock will not be directly exposed to water, handling techniques will depend on the level of acidity of the rock. Low acidity rock may be mixed with a neutralizing material. The material maybe agricultural lime, or possibly on-site soil if it has sufficient buffering capabilities. The mixing procedures typically include placing alternating lifts of rock and neutralizing material. Natural moisture results in the gradual neutralization of the rock. The surface of the fill is covered by relatively impermeable material to reduce surface water infiltration and flushing potential.

For slightly higher acidity potential, it may be necessary to isolate the rock from surface and groundwater infiltration. Isolation of the rock fill reduces the oxidation potential, thereby slowing down the rate of acid production. It is not possible to completely isolate the rock from oxygen and water. However, methods for significant isolation exist. Isolation may include surrounding the acid-producing rock with a neutralizing material (such as lime) and a layer of relatively impermeable soil. Procedures being used in some of the Appalachian states include a minimum 6 ft. thick layer of relatively impermeable material surrounding the acid forming fill.

A second encapsulation method is similar to procedures used for hazardous waste landfills. This method uses synthetic membranes, drains, and soil. One advantage is that an increased volume of rock fill can be used in the embankment because the relatively thin synthetic membrane can meet or exceed the isolation capabilities of several feet of relatively low permeability soil.

For either approach, interceptor or collection drains may also be appropriate to reduce or eliminate groundwater infiltration.

Remedial measures will vary with quantity of acidic rock and availability of alkaline materials and there are no "cook book" solutions. However, a general guideline used by many states is that material with a net acid-base account of 0 to -5 can be blended and treated with agricultural lime and incorporated into normal fill. Net acid base results between -5 and -10 should be encapsulated. Higher values must be handled on a case-by-case basis.

Cut Slopes

Cuts that expose acidic rock may affect surface water quality. However, since the surface area exposed to oxidation is significantly limited compared to a similar exposure of rock fill, less potential exists for a problem with acidic runoff. Also, once the surface of the cut has weathered, there will be little further exposure of fresh rock.

Rock with relatively high acidity may be covered with shotcrete. Fractured, water-bearing strata of pyritic rock may be grouted to reduce permeability, and therefore generation of acidic leachate. These methods are expensive and would generally be done by speciality subcontractors using materials compatible with the acid involved. These types of extreme procedures may also be considered to protect an especially sensitive receptor such as a water supply reservoir.

Handling of Acid-Producing Rock

Multiple handling of acid-producing rock should be avoided. Multiple handling promotes degradation and increases surface area. Increased surface area results in higher potential for acidic leachate. If pyritic rock cannot be placed in embankments immediately after excavation, stockpiles should be located away from surface water, and graded and covered to reduce water infiltration.

As possible, haul roads and excavation schedules should be planned in advance to avoid stockpiling and multiple handling.

If work is suspended during embankment construction with acidic rock exposed, the fill should be sealed with a minimum 6 ft. thick soil cover.

SUMMARY

Iron sulfide minerals oxidize to form acidic, high sulfate, and high iron leachate. Highway construction may accelerate the acid leachate process if iron sulfide minerals are encountered. The resulting leachate can have detrimental effects on the environment and construction materials. The minerals necessary to produce acid leachate can be found in various metamorphic rock units. Review of published geological data and geological reconnaissance can identify these problem rock units. Laboratory tests are required to quantify the acid leachate potential. Testing for total sulfur provides a maximum acidity potential and can identify rock types that warrant further testing. Once the potential is quantified, proper engineering and construction procedures can control production and effects of acid leachate. In some cases, modifications to the highway alignment may be the most appropriate remedial measure.

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QUARRY LAYERS - STRATIGRAPHIC UNITS THAT SERVE THE PUBLIC INTEREST

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ABSTRACT

For over 50 years stratigraphers have mapped and described geologic units on the basis of theories and associations that helped discover petroleum. This quest is often prestigious, romantic, and personally rewarding. The public interest may not be served by mapping for hydrocarbons that, if ever present, are no longer present in the rocks actually used by society. Instead, the service life of our nation's highways and bridges could be improved if the rocks used to produce their PC and AC concrete were better mapped with respect to the engineering properties.

The Code of Stratigraphic Nomenclature has an under-utilized category that would help achieve this goal. This is the quarry layer: an informal, field-recognizable, bed-, member-, or formation-level lithostratigraphic unit. The quarry layer can be subdivided into zones as a further refinement.

A quarry layer can have chemical or physical properties of beneficial or detrimental value to a product. These properties can be determined in accordance with ASTM, AASHTO, or state specifications and procedures. Typical important chemical properties are alkali/aggregate reactivity, electrical conductivity, $CaCO_3$ & $MgCO_3$ content, pH, and chloride content. Important physical properties include gradation, special gravity & absorption, Los Angeles Degradation, and sulfate soundness. Special properties sometimes determined are skid resistance, Atterberg limits, sand equivalency, and particle shape.

Nearly every unit mined for aggregate could be used to illustrate the stratigraphic application of the quarry layer concept. For example, the Sombrerito Fm., Dominican Republic, can be subdivided into 3 quarry layers and several zones that vary into $MgCO_3$ content, water absorption, sulfate soundness, and other properties relevant to production and use of the unit for highway aggregate.

INTRODUCTION

The purpose of this paper is to help implement a beneficial change in the way we collect and present geologic data. Engineering geologists and civil engineers have a problem correlating changes in the physical or chemical properties of geologic units with the stratigraphic breaks defined on geologic maps. The problem stems not from the irrelevance of either classification scheme, but from the fact that stratigraphers are mapping and describing units based on theories and associations that discover petroleum. This is not really a bias, but a direct result of the pervasive influence of the oil industry on academic training, and upon classification schemes (cf. Dunham, 1962; Folk, 1974). It is only natural, since not only is the quest prestigious and romantic, but it also has the potential to be very lucrative. Nevertheless, source- and reservoir-rock concepts are of little relevance to concrete performance, base material failures, or erosion control. And, these are important aspects of rocks that are in the public view. The public interest may not be served by mapping for hydrocarbons that, if ever present, are no longer present

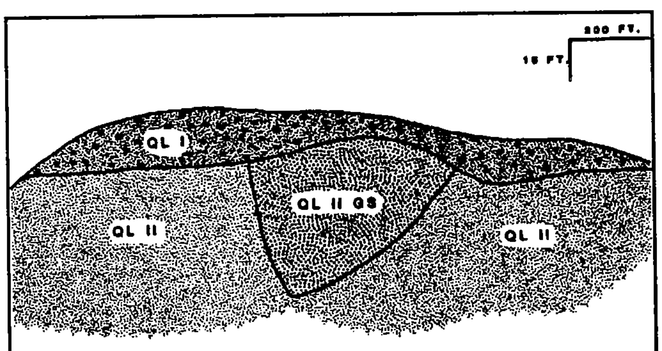

Figure 1: Cross-section of glacial units at Palmer, MA.

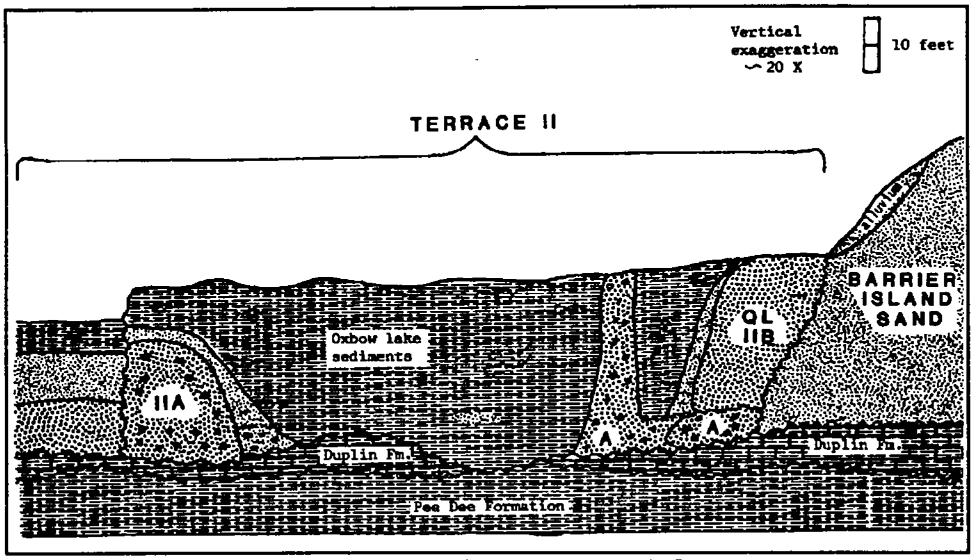

Figure 2: Cross-section of alluvial sand units, southeastern U.S.

changes important to the engineering properties of the rock. With luck, these characteristics would be used more often and eventually supersede the characteristics important to petroleum exploration.

In the Code of Stratigraphic Nomenclature, quarry layers are discussed in Article 22(g); (p. 856). They are: "...in general, informal units even though named. Some such units, however, may be recognized formally as beds, members, or formations because they are important in the elucidation of regional stratigraphy." Perhaps more important is a previous discussion on p. 851, "Some such units are so significant scientifically and economically that they merit formal recognition as beds, members, or formations."

in the rocks used repetitively by society.

Others have also encountered problems with the usefulness of the existing geologic mapping (Banino, 1980; Dever, et al, 1975; Fakundiny, 1980; Hopkins, 1975; Webb, 1970; and Whitson, 1982). In Connecticut, Fakundiny (1980) dealt with a problem he encountered by using a classification scheme of: "Good quality sand and gravel", "Good to intermediate quality sand and gravel", and "Intermediate to poor quality sand and gravel". Banino (1980) used "Cement Limestone" and "Cement Rock" in his effort in Pennsylvania. Hopkins (1975), also in Pennsylvania, used "brickstone". In New Mexico, Whitson (1982) used: "Classical Perlite", "Granular Perlite", "Pumiceous Perlite", along with several variations. Each author chose descriptive terminology but their efforts are not bound together by a common thread.

Perhaps the most effective way to improve the usefulness of geologic maps is to develop uniformity by working within the established Code of Stratigraphic Nomenclature (Amer. Comm. Stratigraphic Nomenclature, 1983). It contains an under-utilized category, the quarry layer, that could allow mappers to be more aware of, and to better recognize the geologic conditions useful to engineering geologists and civil engineers. By using and encouraging the quarry-layer concept, we could begin to improve the way that stratigraphic breaks are defined, and simultaneously elevate the recognition of the physical or chemical

A quarry layer can be defined by chemical or physical properties that are either beneficial or detrimental to an end use. These properties can be determined in accordance with ASTM, AASHTO, state, or other specifications and procedures. Typical important chemical properties relevant to the construction aggregates industry are alkali/aggregate reactivity, electrical conductivity, $CaCO_3$ & $MgCO_3$ content, pH, and chloride content. Important physical properties include gradation, specific gravity, absorption, Los Angeles Degradation, and sulfate soundness. Special properties sometimes determined are skid resistance, Pore Index, Texas Ball Mill Values, Atterberg limits, sand equivalency, and particle shape. This list is not comprehensive and is only given as an illustration.

EXAMPLES

Nearly all units mined for aggregate could be used to illustrate the stratigraphic application for the quarry layer concept. This section will provide some examples involving both unlithified and lithified stratified rocks. Igneous and metamorphic rocks could have similar physical-property-based classifications, but are not addressed in this paper.

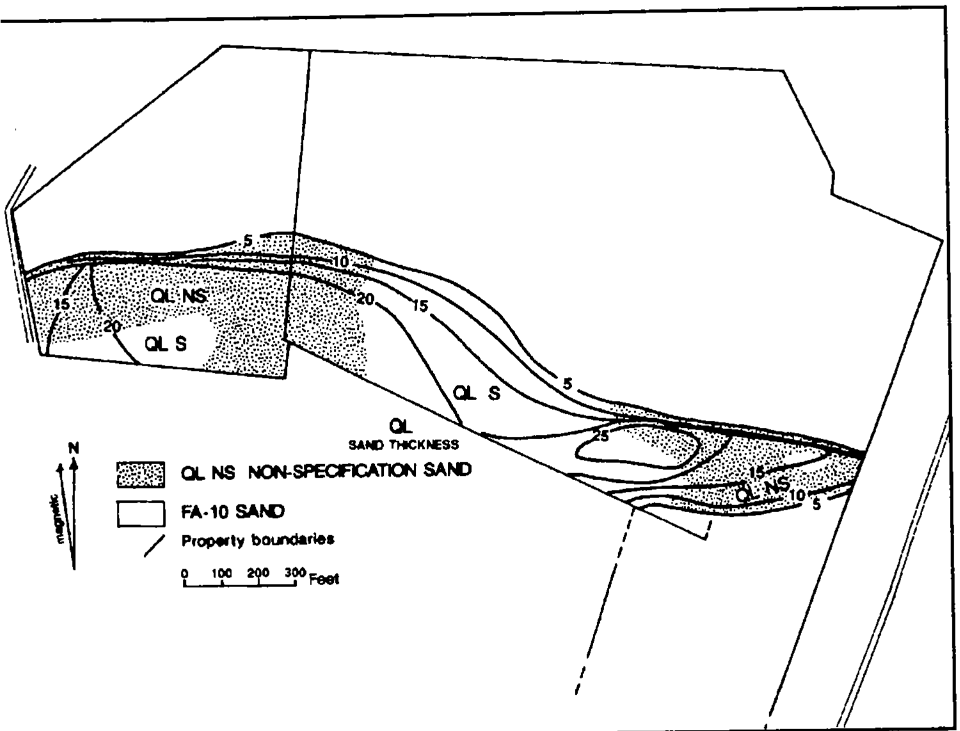

Figure 3: Map of alluvial sand units, South Carolina.

also be used a fine aggregate but, while coarser, has some less desirable properties. It contains shell fragments that affect the overall particle shape of the sand and also decrease its weathering resistance.

Figure 3 illustrates use of the QL concept in a plan view. It is of a terrace sand body in South Carolina (Colquhoun, D. J., 1969; DuBar, J. R., 1971; Thom, 1967, 1970). This sand body is a river bar with two subunits distinguished because of gradation. One (QL S) is a sand that can be excavated and minimally processed to meet FA-10 sand specifications. A gradual change in gradation allows QL NS to be defined. This is a sand that does not naturally meet the FA-10 specifications because of an overabundance in #16 to #50 mesh size material.

Unlithified strata:

Figure 1 is a cross-section through a glacial sand and gravel unit in Palmer, MA. Stone et al. (1979) mapped the unit as "g" (Sand and Gravel Deposits). The cross-section shows lithostratigraphic breaks made on the basis of the gradation of the sediment. The uppermost unit, quarry layer (QL I) is a sand and gravel unit. It has an erosional contact with an underlying sand unit containing incised gravelly sand channels. Note that these channels are indicated as QL II GS. They are the only portion of QL II that could yield a concrete aggregate product, in this case a concrete sand with a minor amount of gravel.

Figure 2 is a diagrammatic cross-section that illustrates a more complex stratigraphic situation. It is from a site in the southeastern United States investigated for concrete sand. This river terrace is a geomorphic/lithostratigraphic unit (Terrace II; Thom, 1967, 1970) that contains two previously unmapped sand units that differ in composition and gradation. They are herein called QL IIA and QL IIB, interpreted as forming in a river bar depositional environment, can be used as a fine aggregate in bituminous concrete. QL IIA is interpreted as a river-channel sand deposit. QL IIA can

Lithified strata:

Figure 4 is a cross-section through a quarry in Kentucky. This quarry is developed in the transitional zone between the Harrodsburg and underlying Borden Formation. Here, units QL I, QL II, QL IIG, and QL IIBT could be defined on the basis of sodium sulfate soundness, Pore Index Values, or lithologic characteristics. QL I is an overall high quality. QL IIG is similar in physical and lithologic characteristics to QL I but is clearly associated with more argillaceous limestones. QL IIBT differs in physical properties from QL II primarily because of a field-recognizable characteristic: it is an argillaceous limestone with a network of bryozoans that improve the apparent quality of the rock. Average values for all units are indicated on the figure. QL I and QL IIG yield products that meet the state specifications. QL II and QL IIBT do not yield products that meet specifications. However, QL IIBT could be produced with QL I or QL IIG to yield some acceptable products.

Furthermore, if QL II is quarried in conjunction with QL I, QL IIG, QL IIBT or any combination, the resulting

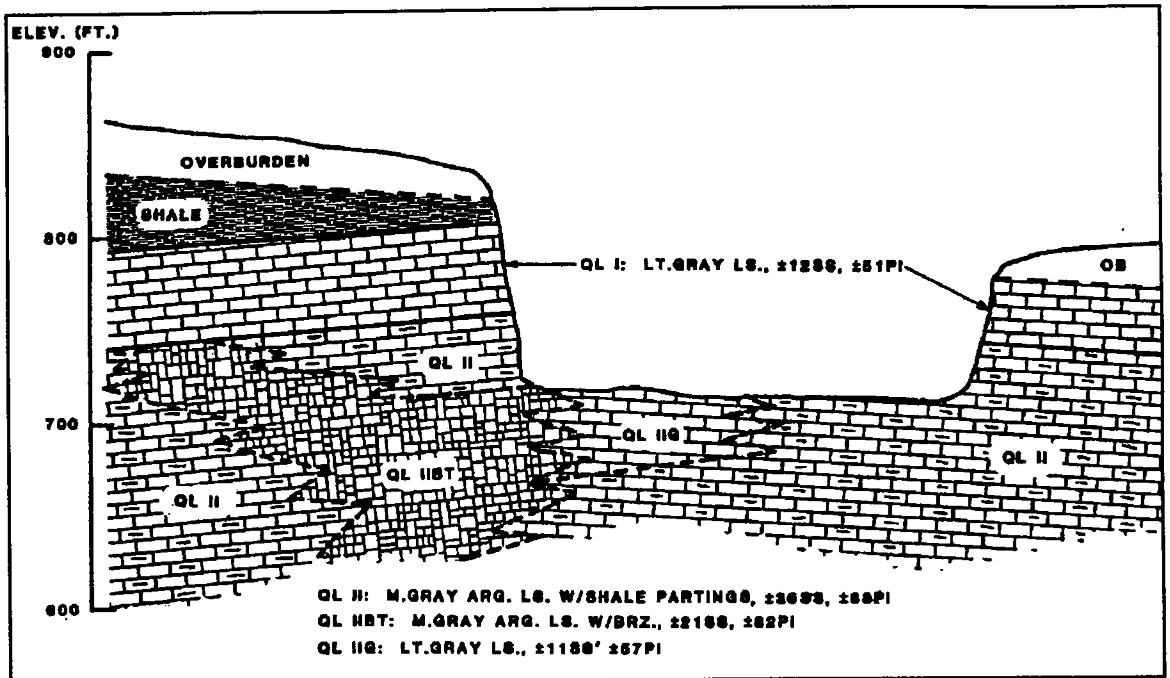

Figure 4: Cross-section of a limestone quarry, KY, (SS=sodium sulfate soundness).

crushed stone products do not meet any state specifications!

Figure 5 is a cross-section through a proposed quarry in the Sombrerito Formation, Dominican Republic (Stokowski, in press). Three quarry layers are present in this marine limestone; they can be further subdivided into zones on the basis of lateritic weathering or dolomitization. The units were selected because of the results of rapid tests performed on drill core. These tests were: color, the apparent strength of the rock based on tapping with a small hammer, the rate of reaction with dilute HC1, and the relative rate of absorption of water by the material. More extensive laboratory testing was also performed (Stokowski, in press). Perhaps the most important relationships are that the non-weathered units (I, ID, II, IID, and III) decrease in coarse aggregate yield and quality with depth and dolomitization. Dolomitization typically causes about 5% lower yield of coarse aggregate, a sand equivalent of about 20 units below the original value, an increase in the water absorption by about an additional 3%, an increase in the LA by an additional 13%, and a decrease in the average 5-cycle $MgSO_4$ soundness with an additional 17% loss.

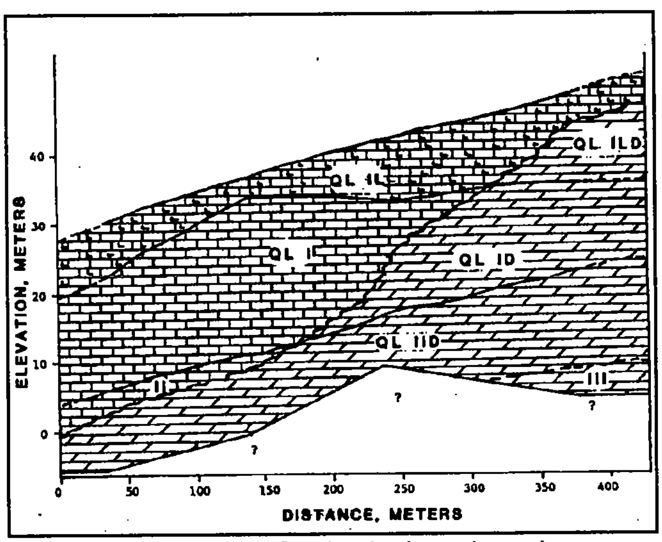

Figure 5: Cross-section, Sombrerito formation units, Dominican Republic. *(Modified from Stokowski, in press).*

CONCLUSIONS

The term quarry layer is useful to geologists and engineers. A quarry layer is an information, field-recognizable, bed-, member-, or formation-level lithostratigraphic unit. The sub-divisions can be based upon chemical or physical properties of beneficial or detrimental value to a product such as construction aggregate. Usage of the term quarry layer would encourage: 1) Uniform terminology during mapping for engineering materials purposes, 2) Definition of stratigraphic breaks because of the physical or chemical changes important to the engineering properties of the rock, 3) Better recognition of the geologic conditions important for quality materials, and 4) Better recognition of the geologic conditions important to engineering geologists and civil engineers.

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Roadways in Karst Terrane

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ABSTRACT

Cambro-Ordovician carbonates are found in many of the Appalachian Valleys of the eastern United States. These ancient rocks, far different in geotechnical characteristics than their well-known but geologically recent cousins of Florida and the Bahamas, can present a variety of poorly recognized problems to roadway designers. The actual subsurface concerns can be exacerbated by:

1. The variety of local names attributed to physically similar formations by state and Federal geological surveys.

2. The difficulty in pursuing effective field investigations economically.

3. The lack of recognition of available construction "fixes."

4. The lack of competent, experienced field supervision during construction.

The first step in any investigation is to review the available geologic data. This information should be used early in the planning stages or the route selection portion of the investigation. A stratigraphic correlation chart for a number of northeastern states has been prepared; it helps the designer obtain and evaluate that information from a variety of sources and relating it to the designer's own experience.

Any investigation must recognize that weathering of the bedrock has produced a residual soil from the parent rock. It must also take into account abrupt changes in subsurface topography and physical properties, as well as open and clay-filled cavities. From an engineering standpoint, the existence of voids in the residual soils above cavernous limestone or dolomites is perhaps more critical than cavity formation or enlargement in these 400 million-year-old rocks. From a practical viewpoint, the detection of small voids and the many irregularities in the subsurface is essentially limited to interpretation of aerial photographs, site reconnaissance, and test drilling.

A number of viable support alternatives are available to the designer. The first is to avoid the area of concern, if possible. The other alternatives are realistic, economic repairs of the subsurface, which may include excavation to rock and sealing the cavities revealed, caissons or piles to sound rock for elevated structures, dynamic compaction, and grouting (in a variety of forms).

The authors cannot overemphasize the need for competent, experienced field inspection. In karst terrane, perhaps more than in any other geologic condition, it is necessary to supplement the information developed from preconstruction investigations with competent on-site inspection. The inspection team should be familiar with the range of subsurface conditions that can exist, and they should be capable of interpreting the possible influence on construction of the variable subsurface can cause.

All solutions and procedures, however, require an early awareness of the geological nature of the problems, a geotechnical understanding of the existing subsurface conditions, and close liaison within the design and construction teams.

INTRODUCTION

Within the Appalachian Mountain chain of the eastern United States, a roadway is best constructed in the valleys, crossing ridges only when necessary. Unfortunately, many of the valleys are underlain by solution-prone carbonate rocks, which make conventional investigative procedures and construction concepts unreliable. Some 20% of the U.S. is underlain by karst. For this paper, however, the authors will primarily address the 300 to 500 million-year-old limestones and dolomites that are found in the Appalachian valleys (see Figure 1). The solutioned nature of these crystalline rocks cause different problems than those associated with the more familiar, geologically recent limestones of Florida and the Caribbean. The geologic origin and

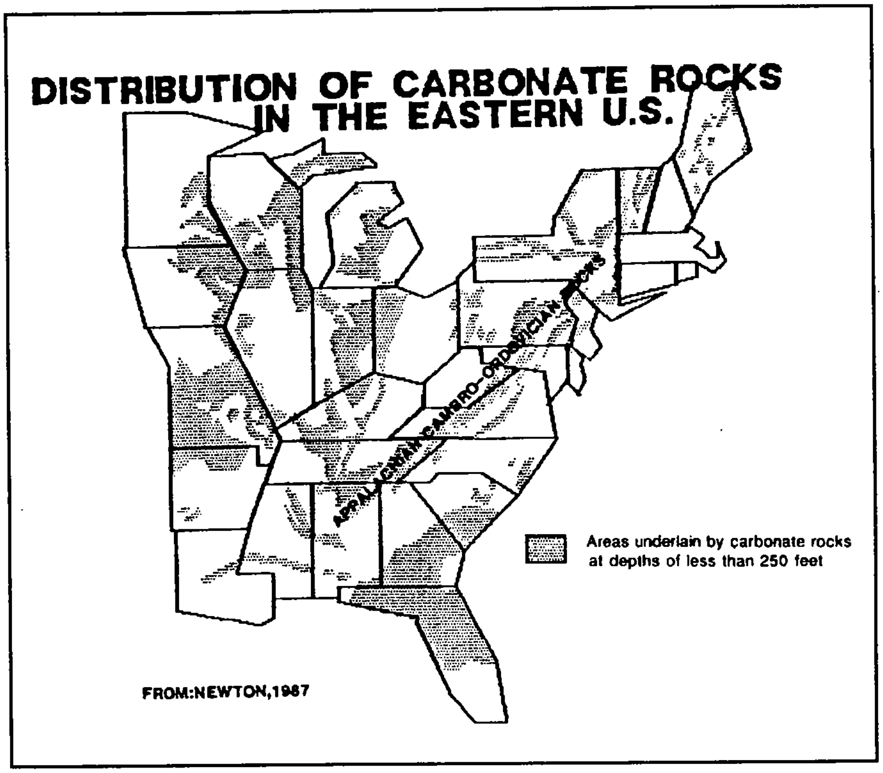

Figure 1:

physical characteristics of these old Appalachian carbonates are different from those of their relatively new southern brethren, and so are the engineering solutions required for safe and economical construction.

The parent materials of the Appalachian carbonates were deposited in a warm inland sea, buried, and then subjected to intense tectonic stresses and weathering (including solutioning). Today, they continue to experience weathering (including nominal solutioning) and erosion. The destructive effects of nature can be, or have been, enhanced by man's activities. Also, it is likely that the intense tectonism experienced in the Appalachians because the region was near an ancient continental margin, exacerbated the susceptibility of these rocks to solutioning over their similar, but apparently less solutioned brethren in the mid-continent.

The same tectonic forces probably helped expose Precambrian-aged marbles (metamorphosed limestones and dolomites) that exist in the region to solution. Fractures in these ancient rocks allowed water infiltration. The water often removed many of the hydrothermal minerals that may have intruded fault and/or fracture zones, as well as removing, by erosion, the stressed, weakened rocks and fault breccia. Hence, even some metamorphic rocks have been solutioned in the past. The voids left behind range in size from minute open fractures to immense caverns. These voids also must be considered by the highway designer.

For most highway projects, however, the designer must consider other factors as well as the existence of caves in the rock. If the planned construction does not markedly increase the subsurface flow of water and if this flow is not rich in carbon dioxide and has a low pH, the rate of solution and erosion will be slow in relation to the lifetime of conventional structures. Thus, it is unnecessary to allow for increased dissolution or erosion of rock in roadway design and construction for a normal economic life. From the stand-point of rock mechanics, it is then necessary only to consider the increase in loading upon existing rock cavities caused by roadways and structures, or the removal of structurally sound limestone over cavernous or weakened zones.

However, a layer of colluvium, residual materials, glacial or alluvially derived soils lying above the solutioned carbonates adds to structural concerns. Percolation of surface water can carry these otherwise sound, overlying soils into solutioned zones within the rock. This movement of material can create open, arched cavities within the soils above the rock. Such an arch can fail as a result of; 1) continuing natural erosion, 2) accelerated erosion caused by construction, 3) the reduction in thickness of the arch because of excavation, or 4) an increase in loading. When failure happens a sinkhole results.

A sinkhole generally develops relatively slowly (if not hurried by man's actions), perhaps over years to tens of years. However, cavern collapse may be virtually instantaneous, with little warning.

Rock cavities are also of concern from an environmental standpoint. Potential for contamination of drinking water supplies cannot be overlooked, particularly in rural areas. The open conduits in karst areas do not allow the natural filtering or adsorption that occurs with conventional fluid movement through soils, and the protection sometimes offered by the relatively slow movement of fluids through most soils is lost. Furthermore, the very large flows of water found in underground openings in carbonate rock formations make these strata a primary target of water well exploration. Hence, any danger to ground water in highway construction is magnified when the roadway and structures traverse karst terrane.

Another concern in construction in some rural areas can be the interuption of normal storm water runoff channels, which sometimes funnel large surface flows into existing sinkholes. While the authors do not necessarily recommend this storm water management procedure, we are aware of its relatively common use in some Appalachian areas. Interruption of storm water flow of this nature has created short-term lakes of considerable extent.

In summary, in order to protect the environment and design safe structures, we must concern ourselves with voids of varying sizes that have formed within the underlying rock over some 300 million years, and with related voids within the overlying soils that have formed over a relatively short time, geologically speaking. In the development of both investigation procedures and engineered solutions, these different aspects of the problem must be recognized. The use of conventional designs and construction methods will increase danger to the public as well as long-term maintenance costs. Therefore, and because of the variable nature of the karst, the authors generally utilize a multi-phased investigation, design, and inspection procedure, as will be discussed.

GEOTECHNICAL STUDIES

Phase I - Preliminary Investigation/Planning

In concept, Phase I is no different than any other well-thought out highway geotechnical investigation. Preliminary geological information is available from various sources such as the U.S. and state geological surveys. Unfortunately, many of the carbonates of concern, which were deposited and consolidated in similar tectonic environments, often receive different names. To aid in correlating the readily available data, Figure 2, which lists a number of stratigraphic correlations, can be used. The information that is likely to prove useful in karst terrane investigations and that is commonly available from the Federal and State agencies can include:

1. Detailed maps (Not always published, but some state surveys are actively involved in mapping areas of carbonate rocks and known or suspect sinkhole areas).

2. Information on well yields by formation type. (Generally, the larger the yields, the greater the void spacing).

3. Data on percentage of voids in sound rock in various different strata. (Some state surveys have compiled such data).

4. Information on the existence of mapped caves and disappearing springs and streams.

5. Information on textural classifications and chemical constituents of the rocks of interest.

6. Information on ground water chemistry.

7. Rock strength data.

Satellite imagery can sometimes be of assistance, and infrared seems to be the most useful. The review of black and white aerial photography is a must in the first stage of the investigation. Inexpensive and exceedingly useful, all available aerial photographs should be obtained from every time period(s) available. On these photographs, karst features may be seen to develop over decades from a slight shadowed depression to a full-fledged sinkhole. Thus, a series of photographs often show a progression of features, perhaps intensifying with time or demonstrating a direction or pattern. Some features that are visible in an earlier photograph, under appropriate moisture or light conditions, can be obscured in later photos by, for example, drying, development, vegetation, or in the case of farm land, the farmer's attempt to maintain a uniformly graded, workable field. Lineaments and circular shapes, observed over time, in a number of photographs should be viewed with suspicion.

The available geologic information and the aerial photographs can be used as the basis for a geologic reconnaissance of the possible route. Linears and suspect areas should be "ground-truthed." For example, in areas underlain by solution-prone carbonates, areas that farmers

Cambrian-Ordovician Geologic Formations of Selected Eastern States

Region of Occurrence of Rock Units	Ohio: Southeastern Northeastern North-Central East-Central West Virginia: High Plateau	West Virginia: Southeast Valley and Ridge Southern Coal Basin Basin Center	West Virginia: Northeast Valley and Ridge Virginia: Northeast Valley and Ridge Rockbridge County Botetourt County Pennsylvania: South-Central Maryland: Washington County	Pennsylvania: Southwestern Northwestern North-Central	Pennsylvania: Eastern New Jersey: Northwestern	Pennsylvania: Northeast New York: West-Central East-Central Eastern	New York: Southeastern
Ordovician	Juniata Formation, or (Queenston Shale and undifferentiated shale and limestone in Ohio) Trenton Limestone "Black River" Limestone or "Black River" Group in NE Ohio) Wells Creek Formation or Chazy Limestone Knox Group (Beekmantown Formation in West Virginia)	Juniata Formation Martinsburg Formation Trenton Limestone Black River Limestone "Wells Creek" Formation ("St. Paul" Limestone in S.E. Great Valley) *Beekmantown Formation*	Juniata Formation Oswego Sandstone, or Bald Eagle Formation (South Central PA) Martinsburg Formation Trenton Group (WV) Chambersburg Group (Black River Limestone in WV) or Chambersburg Limestone (Linden Hall Formation Snyder Formation and Hatter Formation in PA) Liberty Hall Formation *St. Paul Group* (New Market Limestone and Row Park Limestone) Lincolnshire Limestone *Beekmantown Group* (Pinesburg Station Dolomite, Rockdale Run Formation, and Stonehenge Limestone), or Beekmantown Limestone (VA)	Queenston Shale Juniata Formation Bald Eagle Formation Oswego Formation Reedsville Formation Antes Formation Utica Shale Coburn Formation Solona Formation Nealmont Formation Benner Formation, or Linden Hall Formation (North-Central PA) Hatter Formation Loysburg Formation Shadow Lake Formation (Northwestern PA) *Beekmantowm Group* (Bellefonte Formation, Nittany Formation, and Larke Formation, or Stonehenge Limestone)	Martinsburg Formation Jacksonburg Limestone (NJ) or Jacksonburg Formation, Hershey Formation, or Meyerstown Formation (Eastern PA) Annville Limestone Wantage Formation *Beekmantown Group* (Onteiaunee Formation, Epler Formation, and Stonehenge or Rickenback Formation)	Queenston Formation *Lorraine Group* Schenectady Formation, or Frankfort Formation (Eastern NY) Utica Formation Snake Hill Formation (Eastern NY) Martinsburg Formation (Northeastern PA) *Black River Group* Loysburg Formation to Beekmantown Group, undifferentiated (Northeastern PA)	Quassaic Formation Snake Hill Formation Balmville Formation *Wappinger Group* (Copake Limestone, Rockdale Limestone, Halcyon Lake Calc-Dolostone)
Cambrian	*Knox Group* (Copper Ridge Dolomite, or "Mines" Formation in WV) Kerbel Formation Conasauge Formation Rome Formation Tomstown Dolomite Mt. Simon Sandstone, or basal sandstone (Lower Cambrian rocks unknown in High Plateau of WV)	Copper Ridge Formation Conasauge Formation Rome Formation Tomstown Dolomite basal sandstone (Rocks older than Ordovician unknown in Southeastern Great Valley)	*Conococheague Group* (Shady Grove Formation, and Gatesburg Formation), or Conococheague Formation Elbrook Formation Waynesboro Formation Tomstown Formation *Chilhowee Group* (Shady Dolomite, Antietam Formation, and Harpers Formation) Weverton Formation	Gatesburg Formation Warrior Formation Potsdam Sandstone (Northwestern PA) Pleasant Hill Formation (North-Central PA)	*Conococheague Group* (Richland Formation, Millbach Formation), or Allentown Formation Leithsville Formation Hardyston Formation	*Beekmantown Group* (Little Falls Formation, Theresa Formation, and Potsdam Formation) (Rocks older than Ordovician unknown in Northeastern PA)	*Wappinger Group* (Briarcliff Dolostone, Pine Plains Formation, Stissing Formation)

The American Association of Petroleum Geologists, Alen Lindberg Chart Editor.
The above chart is not intended to show strict correlations.

(Modified after: correlation of Stratigraphic Units of North America (COSUNA) Project, Northern Appalachian Region, 1985.
Units appear in appropriate geologic peroid and in superposition.

ıvoid cultivating usually represent rock outcrops or ›ersistent sinkholes. Forested areas in otherwise :ultivated or cleared sections can indicate shallow rock. /egetation changes across a field can sometimes indicate ncipient sinkhole activity. Also, persistent sinkholes are ›ften used as local garbage dumps.

The signatures of possible future problems are many and varied in many karst areas. An initial study can be simply and economically accomplished in almost any ocale. There is no reason to plan a route or start design without completing such a low-cost evaluation of the readily available information.

Phase II - Field Investigations

The results of the Phase I investigation and the requirements of routing, roadway support, structures, and any underground utilities should be considered in planning the Phase II field work. Although the geotechnical investigation can be phased and can vary, in general, the following must be addressed:

1. Location, distribution, and dimensions of rock cavities;
2. Location, distribution, and dimensions of soil voids;
3. Depth and configuration of the rock surface;
4. Variation in physical characteristics of the subsurface soils and rock; and
5. Groundwater quality.

Many direct and indirect procedures have been advanced for the detection of subsurface cavities that have not yet manifested themselves at the surface. These include a host of geophysical techniques,including seismic reflection and refraction, electrical resistivity and conductivity, self-potential, ground-penetrating radar, and gravity surveys.

It is difficult to visualize how these geophysical prospecting procedures can answer the many questions that the geotechnical engineer should be asking in areas underlain by carbonate rocks. They are less helpful than direct measurements of subsurface parameters.

For example, pinnacled rock, large boulders in place, the size and nature of soil voids, and the lack of a coherent groundwater table all compromise the usefulness of seismic surveys in most karst engineering applications.

Conductivity and resistivity are useful in evaluating soil conditions and can indicate variations in rock depth. However, decomposed rock will often look like sound rock on the recorded data, ½ inch wide soil voids atop erratic rock surfaces cannot be inferred, inconsistencies in groundwater elevations can confuse the geophysicist, and cavities will not be seen unless they are quite large.

Ground penetrating radar (GPR) may be useful in shallow rock areas. The Cambro-Ordovician carbonates with which the authors are familiar usually weather to a clayey soil. During most of the year the moisture content of these soils is quite high. Conventional GPR becomes ineffective at relatively shallow depths in these clayey, moisture-laden residual soils. However, in areas of shallow rock or sandy soils, the use of GPR by experienced personnel may be quite useful in finding larger rock cavities. Because of the possible great variability in rock depth and nature across a proposed route, the need to understand the pattern of variability, and the difficulty in finding small voids, GPR is useful in most instances only when used in conjunction with "direct" investigation tools.

However, there are times at which these indirect methods can be effectively combined with direct methods of investigation such as test borings, test pits, and probes. Geophysical tools cannot be substituted for carefully drilled test borings, qualified full-time inspection, experienced professional drillers, and large diameter (Nx or greater) double- or triple-tube, split-barrel coring. However, they can be used to help round-out the information gained through such direct exploration procedures and to expand the area that is investigated in a cost-effective manner.

Invaluable information can be gained from a test boring, in examining for example, the water-softened soils immediately above the rock surface. This procedure allows us to note water or air losses at various depths by observing a) the actual clay-filled seams, b) stained joints, c) weathered cavity, or d) joint sides in a full five-foot run laid out in the core barrel, or by watchinq the drill rods fall through 10 feet of void into a soft cavity filling.

The section shown on Figure 3 was prepared from a composite of supervised test pits, borings and pneumatic probes (with interpretations based upon the authors experience with similar subsurface conditions). It is an example of the variations that can be expected in a short horisontal distance. A closer boring or probe spacing may have indicated even more variations in subsurface topography and stratigraphy.

The air-trac probes were used as one of the investigative tools to develop the section presented on Figure 3. They can be quite valuable when used in conjunction with test borings, because of their mobility, speed of drilling, and relative economy of operation. Although no core samples can be extracted with them, rock depths and quantitative competency can be estimated when the probe data is analyzed in light of information gained from the nearby test borings. An experienced operator can see or feel major changes in the stiffness of the overburden soils as well. Noting the depths of air loss also aids in understanding the subsurface. It is particularly useful when the air loss in one probe hole returns through another.

All drill or probe holes should be grouted to eliminate any new water channels into the subsurface. In addition, maintaining a record of grout-take and related depths often provides useful design information. Equally informative can be the lack of ability to grout a test hole by means of a fluid grout, thus necessitating the use of less conventional procedures (e.g., holes stuffed with cement sacks and then grouted with a dry mix) to prevent water infiltration into the subsurface.

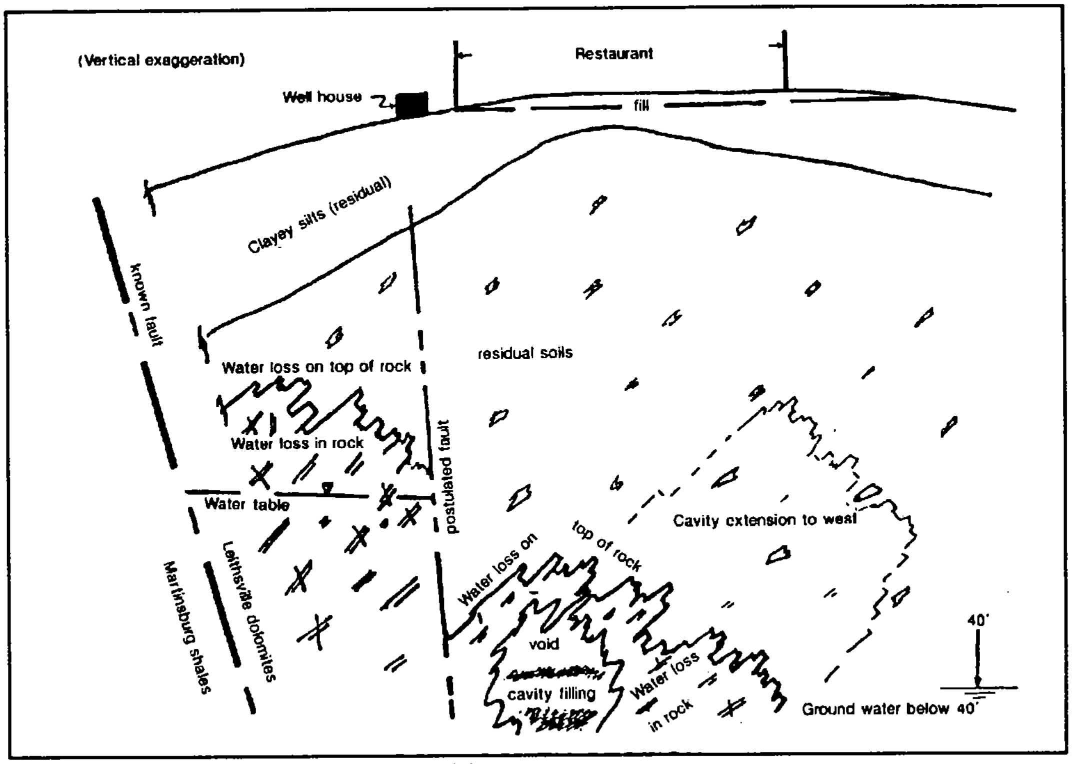

Figure 3: North-south section (not to scale).

CORRIDOR PLANNING

The safety and economic risks associated with roadway development in karstic regions can be eliminated if the alignment of the roadway is adjusted to avoid areas underlain by solutioned carbonates. Geotechnical personnel or firms with extensive experience in investigation and planning in karst terrane should work closely with members of the planning team during the route selection process. The available geologic information is not always precise enough for engineering uses. These geological/geotechnical personnel should be able to identify suspect karst areas within the potential roadway corridors by the relatively inexpensive methods described above under Phase I. Based upon the findings of the Phase I work, the planned routing of the roadway may be altered to avoid a suspect area altogether, or minimize the distance that the roadway must travel within a karst area. In the event that the roadway must traverse a karstic area, the geotechnical personnel may be able to identify the less suitable portions of the route so that those areas may be avoided or better investigated, or the remediation costs estimated.

PRELIMINARY DESIGN AND FIELD INVESTIGATIONS

After the selection of a route that minimizes the risk involved in traversing a karstic area, the design can progress to preliminary alignment geometry and grade. Again, competent geotechnical personnel should be involved in this aspect of the design phase to assist the designers in avoiding or remediating suspected or known trouble areas (e.g., areas that pond water, subsidence features, linears, etc.).

Upon preliminary development of the roadway geometry and grading, those portions of the roadway passing through karstic areas should receive a more intense subsurface investigation program than is conventionally performed, in accordance with the Phase II concepts previously discussed. Although the types and extent of investigation recommended by the authors exceeds the typical roadway geotechnical evaluation, it is our opinion that the information gained minimizes the risk to public safety and reduces the costs of long-term roadway maintenance. Thus, the benefits will likely far outweigh the cost of the investigation.

DESIGN PROGRESSION

With a preliminary roadway layout and the results of the field investigations, the geotechnical and design personnel can initiate roadway design in light of known or suspected subsurface conditions within the karst areas. The geotechnical engineer should now have sufficient information to assist the highway designer in choosing measures for ground improvement, highway design, embankment details, pavement boxes, and drainage structures that would minimize karst related-hazards.

Precluding not cavern collapse, but the migration of overburden soils into the openings (solutioned joints, channels and cavities) within the rock is the most common design objective in preventing sinkhole development . As previously noted, soil migration is most often caused by the movement of water into the underlying rock. The movement of soil generally starts immediately atop the rock and progresses upward as more and more soil is carried into the open rock structure. This process manifests itself on the ground surface as a depression and eventually, if the internal erosion continues, as a sinkhole (doline). However, it must be emphasized that because of variations in overburden properties and thickness, depressions may not develop or may be so small or of such short duration that a sinkhole forms catastrophically with little previous warning.

In preparing the designs for roadway and highway construction, the designers should attempt to eliminate sources of near-surface water accumulation that might cause repeated or continued percolation into the underlying carbonates. Some major design considerations are as follows.

1. Designing impermeable linings for drainage swales and water courses.

2. Using full depth asphalt designs for pavement boxes.

3. If conventional flexible or rigid pavement designs are used, lining the subgrade/subbase interface and the sides of the pavement box with an impermeable material in conjunction with subbase drainage, using drainage tile discharging into catch basins to reduce water infiltration below the road surface.

4. Using gasketed joints on storm sewers and drainage pipe.

5. Not bedding utility pipes, storm sewers, or culverts in crushed stone or other permeable material. The use of indigenous materials, adjusting the structural requirements accordingly, is recommended.

6. Placing the discharge locations of storm sewers far from the highway or nearby structures.

Prior to the construction of roadway embankments and pavement boxes, various ground improvement methods can be used to locate areas of possible sinkhole formation and minimize the potential of sinkhole development. One such method, used by the authors, is dynamic compaction (or perhaps more appropriately, dynamic destruction).

Dynamic compaction involves dropping a substantial weight (on the order of 10 to 15 tons) from heights up to about 60 to 100 feet. The resulting impact can collapse developing sinkholes and identify areas of soft material that fills residual cavities within the overburden. The ground pattern of deformation developed during dynamic compaction can indicate many areas of active and potential sinkhole development.

Once these unstable areas are identified, various remedial methods can be employed. These methods include the following:

1. The overburden soils can be excavated to rock (if shallow enough for the available excavation equipment), to reveal the opening into the underlying rock voids. Then the opening can be sealed with concrete or rock plugs and the excavation backfilled with controlled fill placement.

2. In very shallow rock areas (a few feet or less) and with a highly fractured and solutioned rock surface, the area can be stripped, then sealed by slush grouting.

3. Where rock is deep and the area of concern is widespread, injection grouting through bore holes, executed in an ever decreasing grid pattern, can successfully plug the majority of the solution channels or entrance holes. A cement grout, tailored to the specific application through the addition of accelerators or thickeners is typically used for the initial, secondary, and possibly the tertiary grouting patterns.

4. For support of subsurface utilities in highly variable karst terrane, the authors have utilized injection grouting of both the overburden and the rock underlying the utilities or placing small cast-in-place (grouted) piles.

These methods can be used in any combination and extent or in conjunction with other methods of ground improvement to reduce the degree of risk and the cost of maintenance in most karstic areas.

HIGHWAY STRUCTURES

For highway structures, a number of viable foundation support solutions are available. Dynamic compaction, and excavation and backfill, as previously described for pavement areas, may both be helpful. Transfer of the loads to sound rock is probably the most used alternate. The problem then becomes defining where the rock is sound or how great a depth of sound rock exists over a cavity of what size.

A more complete treatment of various methods for establishing design bearing pressures in karst areas is given in References 1 through 9.

CONSTRUCTION INSPECTION

However careful the preliminary geotechnical investigations and roadway design, one must assume that variations between anticipated and actual conditions will be found during roadway construction in karst areas. For this reason, a geotechnical engineer experienced in design and construction within karst terrane should be available to inspect any of the ground modifications and subgrade preparations. The ability of the geotechnical engineer to observe the subsurface conditions revealed during construction and to alter the ground modification or construction program accordingly can significantly improve both long-term safety and maintenance costs.

Summary and Conclusions

The annual cost of sinkhole repair in the eastern United States is probably in the tens of millions of dollars. Much of this cost, and the possible loss of lives in future construction, can be reduced by 1) a knowledge of the nature of the failures that can occur in karst areas, 2) knowledgeable field investigations, 3) appropriate planning, and 4) experienced construction inspection. Geotechnical tools that reduce both risk and cost are available to highway designers. This paper presents common sense procedures and concepts gained from a large number of good and bad experiences.

Simply, one should:

1. Understand the route's geologic conditions.

2. Understand the planning, engineering, and construction techniques suitable in karst terrane.

3. Utilize experienced construction personnel who can recognize variations in the subsurface (which will be different from expected conditions) and who will be able to institute construction fixes when necessary.

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HIGHWAY CONSTRUCTION IN KARST TERRANES: AVOIDING AND REMEDIATING COLLAPSE FEATURES

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INTRODUCTION

Approximately 20 percent of the land area of the conterminous United States is underlain by carbonate rocks, either limestone, dolomite or marble (Quinlan and Ewers, 1986). Areas of karst topography occur in these terrains, which are characterized by underground caverns, sinks, and other collapse features.

The bulk of these carbonates were formed as marine sediments. However, once these rocks are exposed to subaerial erosion (as a result of continental uplift or sea level changes) they are subject to solution and can produce void features ranging from small pipes and conduits to immense caverns that can hold a fleet of large aircraft.

In the northeast (which we liberally extend south to include Washington DC), carbonate rocks of concern are of early Paleozoic age (mainly Ordovician, approximately 500 million years old (Ma)) that are exposed in the Appalachian Mountain belt as a result of later faulting and folding. This belt of carbonates extends from the Gulf Coastal Plain as far north as Quebec, as do the potential problems of sinkholes and collapse features.

As a rule, New York State and New England are relatively free from the problems associated with collapse features because the sinkholes were either obliterated, infilled, or covered with a thick blanket of glacially transported materials. The number of sinkholes drops off dramatically in the vicinity of the terminal moraine of the last glaciation (Newton, 1987).

This feature, which marks the southernmost extent of the ice sheet of the Wisconsin glacial age, corresponds to the position of Cape Cod (MA), Long Island (NY), and the path of I-80 in New Jersey and Pennsylvania, and continues westward toward the Ohio River valley.

Nevertheless, sinks large enough to swallow roads can still develop north of the moraine (Beaupre and Schroeder, 1989) in glaciated areas, so engineers and road builders have to be aware of the potential hazards involved with placing roads in areas underlain by carbonate rocks.

Our purpose here is to relate a specific group of problems that arose during and after construction of Interstate Highway I-78 in western New Jersey. At the same time, we wanted to provide enough background to make sinkhole formation comprehensible to readers unfamiliar with the literature on karst problems. For additional information on problems in karst areas, we recommend the following works: Beck, 1984, 1989; Beck and Wilson, 1987; and White, 1988.

REGIONAL STRATIGRAPHY AND GEOLOGIC HISTORY

Previously, many of the lower Paleozoic carbonate rocks in the Appalachians were lumped together as the "Kittatiny Formation", but they have since been divided into 5 separate formations, three of which are mentioned here (Dalton, 1976).

The stratigraphic section for the area (Fig. 1; adapted from Drake, 1967) begins with the Cambrian Age (550 Ma) Allentown Dolomite (Ca), which is about 1700 feet thick. This formation yields the largest number of caves in New Jersey, including the Carpentersville caves, and

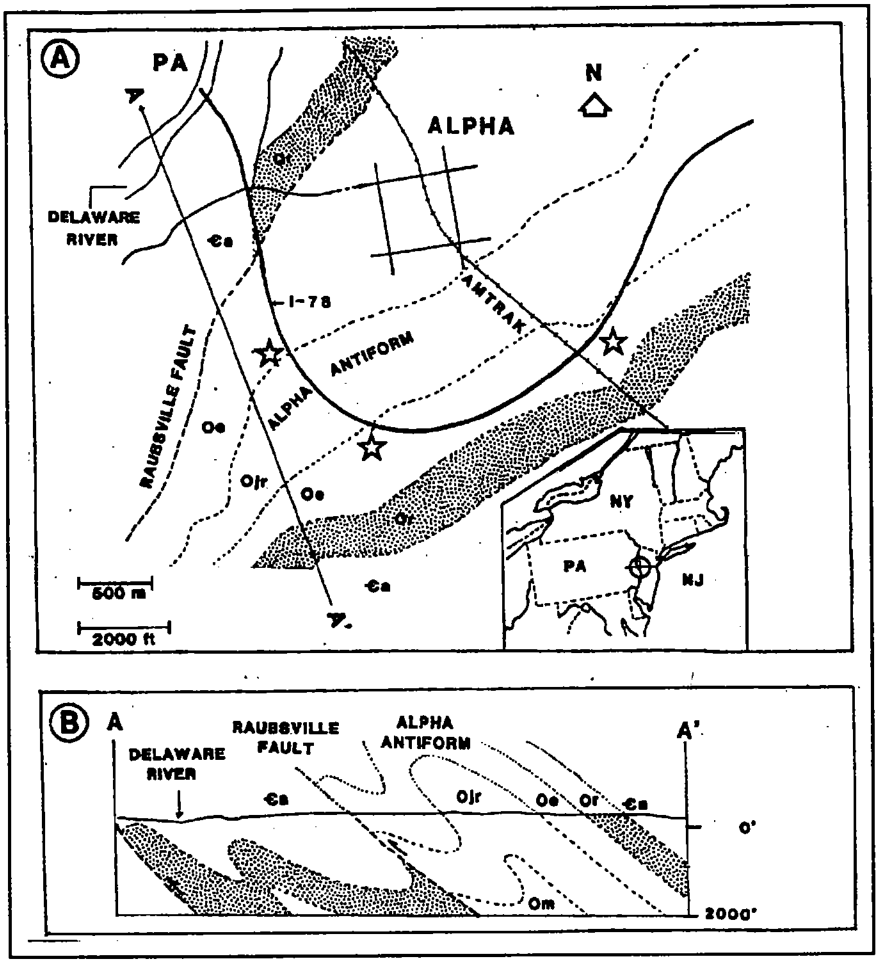

Figure 1:

Map (A) and structure section (B) showing stratigraphic units in relation to I-78 near Alpha, NJ. Stars represent problem areas that developed during and after construction on the route. Legend: Ca, Allentown Dolostone; Or, Rickenbach Dolostone; Oe, Epler Formation; Ojr, Jacksonburg Limestone. Adapted from Mellett and Maccarillo, 1989.

produced some problems in construction of I-78. Above the Allentown is the Rickenbach Formation (Or), a 650-foot thick dolomite that tends to produce sinks where it is exposed in low lying swampy areas.

Overlying the Rickenbach is the Epler Formation (Oe), a unit that is about 800 feet thick, and which has the largest outcrop exposure around Alpha, NJ.

During deposition of the Epler time (470 Ma), you could have paddled a canoe from the Gulf of Mexico to Alaska over a warm tropical carbonate-rich ocean. Following deposition of the Epler however, there was a geologically sudden withdrawal of the limestone-producing seas, and vast areas of North America were subject to a long period (hundreds of thousands to millions of years) of subaerial erosion, during which time substantial karst development took place on top of exposed carbonate rocks. This extensive erosion surface is known throughout North America as the Knox unconformity; it can be found anywhere in the Appalachian region.

Carbonate Formation resumed with the deposition of 650 feet of Jacksonburg Limestone. Following this, muds derived from mountains formed by the Taconic Orogeny produced the thick Martinsburg Shale (Om), and carbonate deposition finally ceased in this area of New Jersey.

HOW SINKS FORM AT THE SURFACE

For the purposes of this paper, we will be referring to any surface depression or collapse feature as a sink or a sinkhole. The subsurface openings in the bedrock below the sink will be referred to as conduits. This may not be good karst nomenclature, but it will be used consistently here.

In dealing with problems in karst areas, it is important to keep in mind that the openings in the carbonate rock (the conduits), whether pipe-like channels or caverns, developed over eons of time, up to tens of millions of years after initial deposition of these marine rocks. The conduits were then often partly infilled with finer-grained material (silts and clays) of younger age.

Thus in Florida, for example, the Eocene Age (40 Ma) Ocala Formation contains silts and clays dated by fossils as Miocene (15 Ma) to Pliocene (5 Ma) age (Beck, 1986). Cave and fissure fillings in Carboniferous age (350 Ma) rocks in the British Isles contain sediments dated by fossils as Triassic (220 Ma) in age. Age of the infilling silts and clays in the Ordovician sediments in New Jersey has not been determined, to our knowledge.

Once uplift occurs and subaerial erosion begins, normally acidic rainwater and carbon dioxide-rich soils, which develop on top of the exposed limestones yield carbonic acid, which leaches out the carbonates and produces the initial process of solution along bedding planes, fractures, or geologic contacts between formations.

In this study area, the most serious problems with sinks have been associated with the contact between the Epler Formation and the overlying Jacksonburg Limestone. This contact is an important one because the top of the Epler is an erosion surface (the Knox unconformity) in which cavernous porosities developed. When the seas covered the surface once again to produce the Jacksonburg Limestone, the openings and conduits in the Epler were never completely filled in, and the porosities exist to this day.

To reiterate, the holes in the bedrock are already there; sinks at the surface form when the sediment plugs collapse or are washed out of the conduits (Fig. 2, A-D), generally as a result of a fluctuating water table.

Human intervention (such as highway construction) often initiates the circumstances that lead to sinkhole formation (Newton, 1987). Many of the sinks would have developed even without human activity to help them along. What we have done, especially since the industrial revolution is to accelerate the natural processes that led to sinkhole formation. So sinks that may have opened up in 1000 to 10,000 years without human intervention in the landscape now begin to open up within tens of years.

Fig. 2A shows a typical area after the subaerial erosion episode that produced the Knox unconformity. Fig. 2B shows younger sediments covering and partially plugging the conduits, after the rocks were folded by later mountain-building episodes (I have not shown them upside down, as they really are in the field). The pre-construction drainage divide is also shown. The position of the piezometric level (the water table) is depicted in the undisturbed state.

With highway construction commencing, the drainage divides are shifted, subsurface drains are put in, and the entire hydrologic regime of the area is altered (Fig. 2C). The water table now begins to fluctuate dramatically, and the buoyant effect of water, which helps support the sediment plugs, is diminished. Subsequently the fine-grained sediments in the conduits begin to ravel and fall downward, to wash away, or accumulate at the bottom of the conduit, which may be 100 feet or more below grade. Eventually the overlying sediment is no longer competent enough to support itself, and the sink opens up at the surface, undermining the highway (Fig. 2D).

SUBSIDENCE PROBLEMS ON I-78

Planning for this 4 mile long segment of the Interstate Highway system began in 1980, and exploration crews encountered immense caverns during drilling operations. This section of I-78 was one of the last to be completed in New Jersey. and the route location was "locked in" because of completed highway segments east and west of Alpha.

A geologist employed by the engineering firm was laid off in a budget cutback prior to the design phase. After discovery of the caverns and voids in the drilling operation, the only contract modifications that were made called for the filling in of surface voids, and a recommendation to carry out a selective seismic refraction survey within the highway cut. Unfortunately, the areas chosen for the refraction did not lie near the problem-plagued Epler-Jacksonburg contact, and were done in areas of relatively sound rock.

Construction began in 1985, and was completed in 1988. The highway opened in 1989. Many problems involving ground subsidence were encountered in that 4 year period, particularly after heavy rains. In addition to sinks opening up alongside the highway, concrete roadbed slabs were offset and undermined, and gabions sagged. There was collateral damage to other structures. A total of 60 sinks were encountered during construction, and 8 since the highway opened.

The most serious incident just before the highway opened involved the discovery of a cavern 50 feet wide and 180 feet long. A sinkhole at the surface opened up in the median, and under two lanes of the highway. Actual subsidence only occurred under two lanes of the highway, although the cavern extended underneath all 6 lanes, both shoulders, and the median. The cavern extended to a depth of 65 feet below grade; it has since been remediated.

After the highway opened, another incident occurred when a police car made a U-turn in the median to pursue a speeder going in the opposite direction. The chase was aborted when the rear wheels of the police cruiser fell into a small sink.

Had the highway taken a more direct westerly route and run through, or north of the town of Alpha, the right-of-way would have gone directly across the Epler-Jacksonburg contact only twice, on the eastern and western limbs of the Alpha Antiform (an antiform is an upside down syncline that looks like an anticline in section). But in a direct verification of Murphy's Law, the highway ended up paralleling the contact for some distance and crossed it four times at oblique angles, assuring a host of problems with road construction.

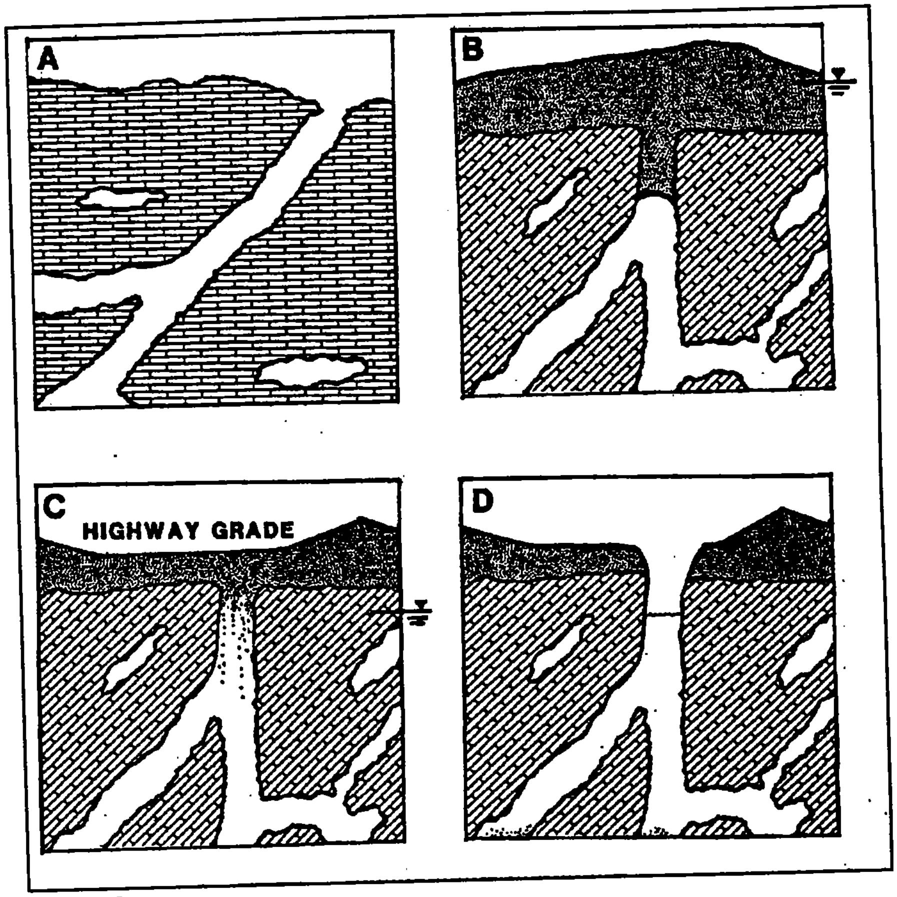

Figure 2:

Profile sequence showing origin and development of sinks in the Epler Formation. (A) Knox unconformity, 470 Ma, showing cavernous porosities developed in rock, (B) Sedimentary or soil covering on Epler rocks after Appalachian folding, and just prior to construction. Local water table shown. (C) New grade for highway in place, with a fluctuating water table. Note fine-grained material in conduit beginning to ravel downward. (D) Sinkhole open at surface.

DEALING WITH THE PROBLEM

There are 3 levels at which one can deal with these problems.

(A) Avoidance of karst areas;
(B) Assessment and preparation;
(C) Remediation.

AVOIDANCE

Avoiding the problem sounds attractive, but it is easier said than done. Prior decisions generally "lock in" the right-of-way, and any subsequent large scale shift of the highway route will become prohibitively expensive. Because northeast-southwest trending limestone belts are exposed in the folded Appalachians, east-west highways that run any distance are bound to cross them. Occasionally where there are isolated outliers of carbonate rock, highways could be routed around them without difficulty, but for the most part, avoidance of karst terrane in the northeast and throughout the entire Appalachian region is not a viable alternative.

ASSESSMENT AND PREPARATION

In assessing and preparing for karst related problems, a variety of remote sensing and geophysical methods, such as ground-penetrating radar, high resolution seismic reflection, electromagnetics, infrared photogrammetry, etc., can be used to detect underground cavities prior to routing and construction of the highway (see Fischer et al, 1987, 1989, and Benson and La Fountain, 1984 for a detailed list of assessment procedures). However, absence of findings from prior surveys cannot be used to assure that future problems will not arise. In many cases. as indicated above, construction of the highway itself initiates collapse features that may only become apparent after the final stages of paving are completed. Hence our later argument for a waiting period after initial phases of construction.

State and local highways in this same region may not have experienced sinkhole problems because they kept drainage changes and cutting and filling to a minimum. Interstates are much wider, have drains on the shoulders and medians, and the highway roadbed is excavated to a greater depth.

REMEDIATION

Remediation of collapse features will generally involve the use of geotextiles, dense graded aggregate, and/or grouting. Where sinkholes are encountered prior to or during construction, the surrounding area should be excavated to sound rock. Conduits can then be located and plugged. Large openings should be lined with geotextile filter fabric, and backfilled with dense, graded aggregate. Use of aggregate rather than an impermeable concrete plug permits drainage to still flow down the conduit. The geotextile is used to prevent the fines from washing out of the aggregate fillings and going down secondary outlets. It is important to excavate these holes down to bedrock; if one does not get the hole filled correctly the first time, subsequent heavy rains may cause the sink to reopen.

It is important to note that of the 60 problem areas that were discovered and remediated during construction of I-78, all remained stable since paving was completed. That is the strongest evidence we have in favor of inserting a waiting period between initial construction and final paving.

The cost of remediation after construction exceeds the cost of remediation prior to construction by a factor of 30. A large sink that required repair after undermining two highway slabs would have cost $15,000 to fix had it been discovered prior to paving. Remediation after paving produced a cost of $450,000. Even accounting for inflationary increases in construction costs over a 2-year period, repairs after paving still greatly exceed those made prior to paving.

Remediation of problems under the roadbed involved drilling a series of 12-inch diameter holes, and filling the voids with a standard concrete mix with aggregate. Smaller subsurface openings can be filled with a grout mix of 2 parts sand, 1 part cement, and enough water to allow the material to flow through a l-inch diameter pipe.

But karst areas are so unpredictable that even remediation of sinks by plugging them up can alter groundwater flow and have an influence on, and induce other sinks many hundreds of feet away.

RECOMMENDATIONS

We recommend the following steps be taken in any highway construction in areas of karst terrane:

(1) Following infrared photogrammetry, lineament analysis from aerial photos, and so on, placement of the route should the be followed by an intensive shallow geophysical survey in order to detect large, capacious voids that might threaten the entire right-of way.

Terrain conductivity surveys (electromagnetics, or EM) are helpful in locating deeper voids, or zones of fine-grained sediments in conduits below the surface. Ground-penetrating radar (GPR) is also helpful, although the clay-rich content of many soils that overlie carbonates often limits depth of penetration of the radar pulses (Fischer et al, 1989). Nevertheless, even where the depth of radar pulses is limited and a deep conduit cannot actually be "seen", the GPR can locate what Benson and La Fountain (1984) call "near surface indicators": soil or vegetation disturbances that may hint at a developing sink below them.

Figure 3 shows radar scans made over the Jacksonburg Limestone on a stable portion of the highway right of way, compared to scans made near the Jacksonburg-Epler contact, where remediation is usually required. Although not shown on these scans, the radar pulses detected targets thought to be potential conduits as deep as 160 nanoseconds (ns), or about 32 feet below grade.

High resolution seismic reflection, and seismic refraction also can clearly delineate potential problem areas. Refraction was useful to contractors in helping them distinguish zones where blasting or ripping was appropriate, but it did little to discover problem areas along the route because the areas chosen for refraction studies were done in void-free rock.

Routine office use of geologic maps in assessment may not be a reliable indicator of an area's susceptibility to karst problems. Elsewhere in New Jersey, for example the Epler Formation is not distinguished by extensive karst features, but it is near the Delaware River around Alpha and Phillipsburg.

(2) Because changes in drainage during and after construction are the chief initiators of sinkhole formation at the surface, cutting and filling should be kept to the absolute legal minimum necessary to comply with state

and federal regulations concerning grades and curve radii.

(3) Drainage divides should be altered as little as possible from their original placement and orientation on the ground surface.

(4) Our most important recommendation is that a two-year lag be placed between initial grading, and completion of the project. After the highway is laid out, initial grading should be completed, and the entire project should be allowed to sit undisturbed for two years prior to final grading and paving. The idle period may not "catch" all the developing sinks, but it may allow 80-90 percent of them to develop. This period of "idleness" allows sinks time to appear at the surface, and be remediated before final paving.

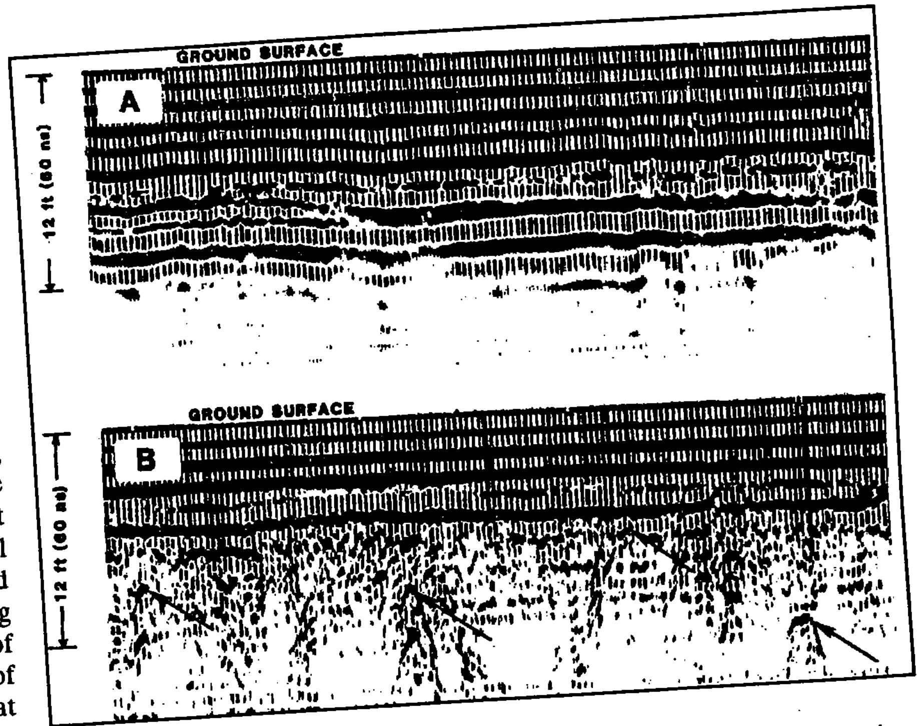

Figure 3:
Ground penetrating radar (GPR) scans on shoulder of I-78. (A) Stable area in Jacksonburg Limestone, which showed no evidence of subsidence of sinks. (B) Area at Jacksonburg-Epler contact underlain by conduits (arrows). Note major difference in radar signatures on the two scans below about 5 ft. under the surface. Width of section about 250 ft. Scans made with Geophysical Survey Systems Inc. (North Salem NH) 100 Mhz transceiver model 769DA. Scans recorded on GSSI model PR-8304 graphic recorder set at 200 ns range.

For example, a waiting period is often used in dewatering large areas of organic materials found along proposed highway construction routes. Previously, draglines could be brought in to remove organic muck and dump it elsewhere, but elsewhere no longer exists, or no longer accepts material for dumping.

A current method for dewatering organic mucks is to surcharge the area with a 40-foot thick mound of fill, and install horizontal and vertical sand drains in it. The weight of the overlying material is then utilized to compress the muck and squeeze the water out of it. In this case, the project may be allowed to sit for up to 5 years before the highway is completed.

Failure to remove or dewater organic materials along a right-of-way can lead to later subsidence problems along the route. We are recommending that a similar waiting period be initiated to reduce subsidence problems for highway construction in karst areas.

Once the surface drainage patterns have been altered, sinks will generally reach the surface and open up after every heavy rain, until the system finally stabilizes to the new hydrologic regime. Once sinks open up, the problem areas can be noted, and remediation can begin.

(5) Finally, after final paving and placement of structures, shallow geophysical surveys should be run annually for at least 3 years and archival records should be kept of all such geophysical surveys in order to monitor sites of potential ground subsidence and subsurface collapse. If a possible open conduit can be seen at a depth of 25 feet below grade, it may not be a threat. But if subsequent scans shows it moving up closer to the surface, then the integrity of the roadbed may be threatened and action will have to be taken to plug the evolving sink.

Geophysical monitoring may have to take place for the life of the project. Once a highway is constructed, one can expect subsequent development nearby. Open farmland will be graded, paved, and drained as shopping malls and housing developments begin to cluster around the new transportation corridor, and once again the hydrologic regime of the area is altered. Because conduits in karst regions can extend long distances laterally, it is conceivable that these adjoining developments may have an adverse impact on a highway route.

CONCLUSIONS

We had originally considered using as a title "How not to build a highway...in a hurry", but opted for a more conventional title. In karst areas, patience is a virtue that will save a great deal of time, anguish, and money in the long run. Letting a major project sit for 2 years in order to allow sinks to appear at the surface is not an approach that engineers, contractors, insurance companies, or federal agencies and municipalities are going to be happy about. But the unique and unpredictable characteristics of karst terrains require unconventional methods for dealing with them, particularly for projects as important as major highways.

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APPLICATION OF NON-DESTRUCTIVE TESTING TECHNIQUES TO SLOPE STABILITY AND SINKHOLE MONITORING

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ABSTRACT

In recent years studies have been underway at the Pennsylvania State University to evaluate the feasibility of utilizing non-destructive testing (NDT) techniques to investigate a variety of geotechnical problems. This paper discusses a number of NDT techniques and describes the application of three specific techniques, namely: the acoustic emission/microseismic method, seismic tomography, and experimental modal analysis to slope stability and sinkhole monitoring.

INTRODUCTION

Many of this country's major highways were constructed more than 20 years ago and are exhibiting increased signs of "old age." Furthermore, the rapidly increasing cost of highway maintenance and repair, and the desire to increase highway safety, necessitates improvements in the related geotechnical procedures. In particular, there is a clear need to upgrade the presently available techniques for evaluating the structural stability of such components as highway road beds, tunnels, rock and soil slopes, and bridge decks, piers and abutments.

In recent years studies have been underway at the Pennsylvania State University to evaluate the feasibility of utilizing non-destructive techniques (NDT) to study a variety of geotechnical problems. Three specific techniques, namely: the acoustic emission/microseismic (AE/MS) method, seismic tomography (ST), and experimental modal analysis (EMA) have been investigated. This paper will first discuss the basic concepts of each technique, the associated instrumentation, and a number of current applications. Following this introductory material the paper will consider in some detail two specific NDT field applications, namely: slope stability evaluation and sinkhole location and stability monitoring.

NON-DESTRUCTIVE TESTING METHODS

1. General

In 1991 the American Society for Non-Destructive Testing (ASNT) celebrates 50 years of service to American industry, however, a variety of non-destructive testing methods were in use well before the founding of this society. Today a wide range of NDT techniques play an important role in the construction, manufacturing, and service industries throughout the world.

A number of NDT methods are available for use in geotechnical applications. These include visual inspection; the use of colored penetrant magnetics, electrical resistivity, ground probing radar, seismic transmission, reflection and refraction; and methods based on ultrasonic, pulse-echo, acoustic emission/microseismic and experimental modal analysis techniques. Further details on the above NDT methods are presented throughout the literature (e.g. Belesky and Hardy, 1986; Hardy et al., 1986; Sun, 1990).

Unfortunately at present, NDT techniques, similar to those noted above, are not routinely utilized in civil engineering, even though they can provide critical data for design and maintenance not available using other less sophisticated techniques. In order to promote the routine use of such techniques, those involved in the field of civil engineering need to become more aware of the available "NDT tools". Furthermore, funds need to be made available to carryout the studies necessary to evaluate their suitability and estimate their current limitations relative to a variety of typical field conditions. A number of such studies have been undertaken by the writer. These include the use of the acoustic emission/microseismic (AE/MS) technique to investigate highway slope stability and sinkhole development; the application of seismic tomography (ST) to evaluate sinkhole location; and the use of experimental modal analysis (EMA) to evaluate the stability of planar rock slopes. In the following sections the basic concepts, instrumentation and current applications of the AE/MS, ST and EMA techniques will be briefly discussed.

2. Acoustic Emission/Microseismic Technique

2.1 Basic concepts

It is generally accepted today that most solids emit low-level seismic signals when they are stressed or deformed. This phenomenon will be referred to here as acoustic emission/microseimic (AE/MS) activity. In essence, the measurement of AE/MS activity in a field structure is relatively simple. A suitable transducer, normally an accelerometer or velocity gage, is attached to the structure, the output of the transducer is connected to an associated monitoring system, and the seismic signals occurring in the structure due to internal or external stresses or deformations are suitably detected, processed and recorded. Figure 1 illustrates a typical example where a number of transducers (an array) are installed to monitor the stability of a highway tunnel.

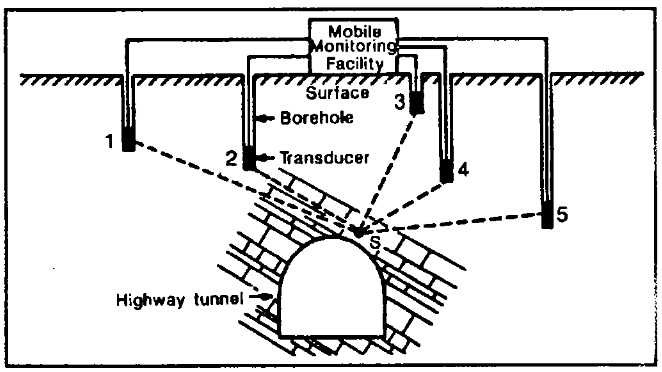

Figure 1: AE/MS transducer array installed to monitor the stability of a highway tunnel. (S denotes the assumed location of an AE/MS event.)

Detailed discussions relative to the fundamentals of the AE/MS technique are presented elsewhere (Hardy, 1981; Miller and McIntire, 1987). In review a number of the more important AE/MS concepts are as follows:

(1) The AE/MS technique is a passive, indirect technique.

(2) AE/MS activity originates as an elastic stress wave at locations where the material is mechanically unstable.

(3) The associated stress wave propagates through the surrounding material undergoing attenuation as it moves away from the source.

(4) With suitable instrumentation AE/MS activity may be detected at locations a considerable distance from its source.

(5) The useable spatial range of the technique is dependent on the frequency content of the source and the characteristics of the media and the monitoring facility.

(6) The character of the observed AE/MS signals provide indirect evidence of the type and degree of the associated instability.

(7) Analysis of data obtained from a number of transducers (array) make it possible to determine the actual location (i.e., spatial coordinates) of the source.

2.2 Instrumentation

General--Figure 2A illustrates a block diagram of a simple, single channel AE/MS monitoring system. It consists of three major sections, namely, the transducer, the signal conditioning unit, and the readout unit. In such a system, AE/MS activity is detected by the transducer, the resulting electrical signals are suitably modified by the signal conditioning section, and finally displayed and/or recorded in the readout section. In general, two totally different types of signal conditioning are in common use; for the purpose of this paper these will be denoted as "basic" and "parametric."

A multi-channel monitoring system is normally required to obtain meaningful field data and a simplified four-channel system is illustrated in Figure 2B. It is important to note that in most cases such a system merely involves additional transducers and signal conditioning units, with the total data from all channels being recorded on a single multi-channel readout system. Regardless of the number of monitoring channels involved, the various components of the system must be

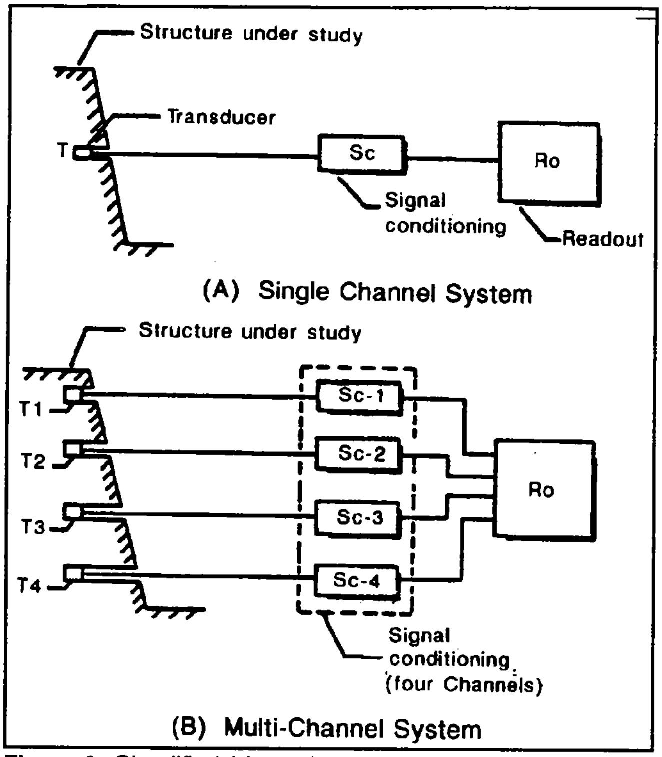

Figure 2: Simplified block diagram for a single- and multi-channel AE/MS monitoring system.

selected to provide the frequency response, signal-to-noise ratio (SNR), amplification, parametric processing, and data recording capacity necessary for the specific study.

Transducers--The purpose of the transducer is to convert the mechanical energy associated with an AE/MS event into a suitable electrical signal. When a geologic structure is loaded or deformed, mechanical signals are generated due to localized deformation, and/or failure at areas of high stress concentration. AE/MS activity at a specific point in the structure may be detected by monitoring the displacements, velocities or accelerations generated by the associated mechanical signals at that point using a suitable transducer.

Where signals containing relatively high frequency components (f > 2000 Hz) are involved, accelerometers are usually employed. In contrast, low frequency signals (f < Hz) are usually detected with displacement gages. Signals between these extremes (1 Hz < f < 2000 Hz) are conveniently detected using velocity gages. Typical geophones and accelerometers have sensitivities in the range 10-100 V/ft/s and 2-100 mV/g, respectively. Although geophones are intrinsically sensitive to P-waves only, accelerometers are available for the direct detection of both P- and S-waves.

Although both geophones (velocity gages) and accelerometers have been used in monitoring such geologic structures as mines and tunnels, in the majority of cases, geophones are normally employed. These transducers are less expensive than accelerometers and are generally more sensitive in the lower frequency range where much of the AE/MS energy associated with large structures occurs.

Transducer Location--Figure 3 illustrates a simplified field situation where it is desired to monitor AE/MS activity associated with an underground geologic structure (ST). This structure, which could be for example a tunnel, underground mine, or a storage area, is assumed to be located at an average depth below surface of d_1. It is also assumed that the site is overlain with a layer of unconsolidated material, for example soil, to a depth of d_2. One possible mode of AE/MS instrumentation would involve installing a suitable number of transducers on the surface of the structure (A) and/or in holes drilled outwards from it (B). This would require access to the structure. A second mode of instrumentation, and one which is in many cases more convenient, or the only one possible, involves installing transducers from ground surface above the structure (Cl-C4).

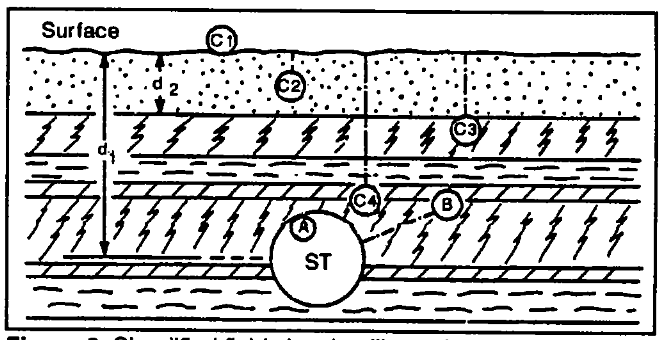

Figure 3: Simplified field situation illustrating various possible locations for AE/MS transducers.

The type of transducer, and the location and depth of the installation, will depend on a number of factors including, amongst others, the depth of the structure (d_1), the depth of the overburden (d_2), the thickness and properties of the strata overlying the structure, the expected energy of AE/MS events to be detected, the desired source location accuracy, and the funds available. It should be noted at

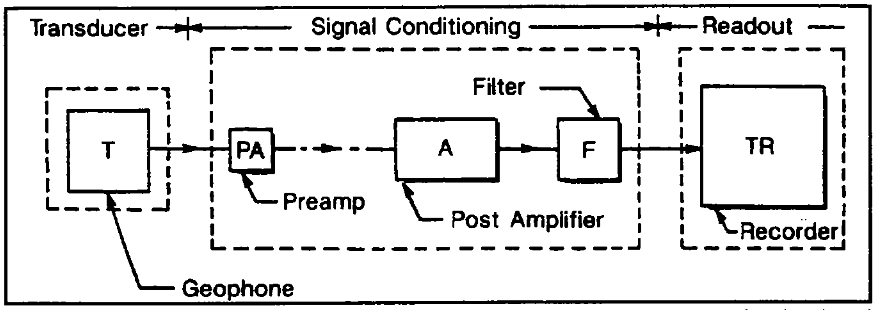

Figure 4: Single-channel AE/MS monitoring facility incorporating a basic signal conditioning system.

this point that there appears to be no one single type of installation which is suitable for all studies; experience has shown that the optimum system must be tailored to the specific application.

Signal Conditioning--Two different types of signal conditioning are commonly in use in AE/MS monitoring facilities, namely, basic and parametric. Figure 4 illustrates a simplified block diagram of a basic system. In such a system AE/MS signals detected by a suitable transducer (e.g., a geophone) are first passed through a preamplifier located near the transducer. A field cable carries the amplified signal to the monitoring facility where the signal is further amplified, filtered, and finally recorded on a multi-channel tape recorder. The important feature of the basic signal conditioning system is that the AE/MS data, although amplified and filtered, still retains much of its original analog form.

In contrast to the basic system, the parametric system incorporates additional signal conditioning facilities which further process the amplified and filtered AE/MS signals to provide a series of specific parameters. For example, Figure 5 illustrates the block diagram of a single channel monitoring system incorporating a parametric signal conditioning system which automatically provides AE/MS event rate data. Such systems are available to provide one or more AE/MS parameters such as total number of events, event rate, event energy, amplitude distribution, etc. For the most part these parameters are determined using digital techniques.

Although such parametric signal conditioning systems are extremely useful in certain specific situations, they have a serious disadvantage in that the original analog form of the AE/MS data is permanently lost. A number of systems utilizing parametric signal conditioning, some containing micro-processor units and capable of multi-channel monitoring, are commercially available.

Readout Systems--Depending on the type of signal conditioning system utilized, a variety of readout systems are available for storage and display of AE/MS data. If basic-type signal conditioning is employed, the most flexible system is one incorporating a tape recorder and an associated strip chart recorder (usually a high speed U-V recorder) for detailed visual examination of the data. Storage-type oscilloscopes and transient recorders are also useful for evaluation of selected AE/MS signals. When parametric-type signal conditioning is employed, the data (such as AE/MS rate) may be displayed on a simple panel meter, or on a digital display, printer, or graphics unit. In many respects the type of readout system employed is intimately related to the method of data processing to be utilized for later analysis of the collected data.

Multi-Channel Monitoring Systems--As noted earlier, systems similar to those shown in Figures 4 and 5 are suitable for monitoring data from only a single transducer. In many field cases, however, the actual source of the AE/MS activity is required and this can only be obtained using data from an array of transducers (normally a minimum of five). Therefore, a multi-channel monitoring system is required. The multi-channel mobile monitoring facility developed at Penn State (Kimble, 1989) is a typical example of such a

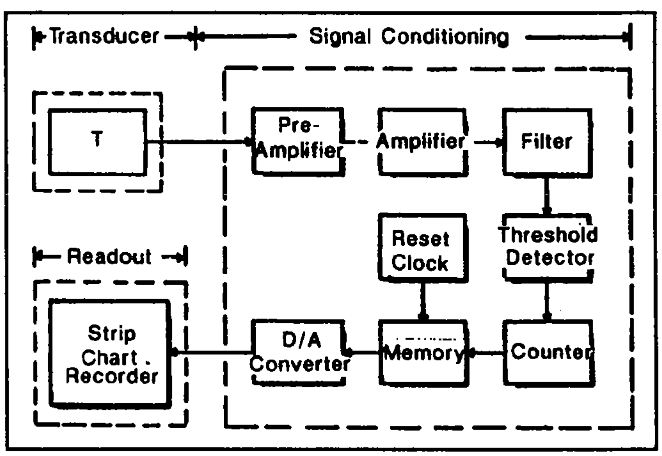

Figure 5: Single-channel AE/MS monitoring facility incorporating a parametric signal conditioning system.

facility. Figure 6A shows a block diagram of the system which was designed to handle up to 14 channels of AE/MS data. Figure 8B shows a photograph of the completed system. System power was provided from a motor generator or from local power (if available), and in order to carry out measurements at a number of different locations, the completed facility was housed in a small truck and an associated trailer.

2.3 Recent Applications

In recent years the application of AE/MS techniques in the general area of geotechnical engineering has rapidly increased (Hardy, 1989A). At present such techniques are in use, or under evaluation, for stability monitoring of underground structures such as tunnels and mines, natural gas and petroleum storage caverns, radioactive waste repositories, and geothermal reservoirs. Surface and near-surface applications include rock and soil slope stability, subsidence, water flow and foundation stability. A number of recent Penn State AE/MS studies associated with various geotechnical applications are briefly discussed in this section.

Cavern Stability--In 1978 AE/MS studies were carried out by the writer in a large scenic cavern located in central USA (Hardy, 1981). Since the area had been closed to the public for some years, and since minor apparent structural instabilities had been noted in the past at this site, it was felt that an evaluation of the safety of the area should be undertaken as a preliminary step in the reactivation of the area for public use. At the cavern test site, transducers were installed on surface and throughout the cavern. AE/MS measurements were made using the earlier Penn State Mark I mobile AE/MS monitoring facility. Over a four-day monitoring period the largest ground motions recorded at the site were considerably less than those generally accepted as the lower limit for induced structural change. It was concluded therefore that during the monitoring period, the site, from a mechanical point-of-view, was highly stable. In recent years the site has been renovated and is now open to the public. No signs of structural instability have been observed to date.

Longwall Coal Mine Studies--During the period 1970-1977 the Penn State Rock Mechanics Laboratory was involved in field studies undertaken to investigate the feasibility of using AE/MS techniques to locate potential zones of instability around underground coal mine workings. This study was unique in the fact that measurements were made from the surface rather than underground. When this project was initiated in 1970, little or no scientific evidence existed to substantiate the fact that AE/MS activity generated underground, due to routine longwall mining activities, could be successfully detected by near-surface transducers.

At this site an array of 15 transducer sites were located directly over an active longwall panel located some 500 ft below surface. The Penn State Mark I mobile AE/MS monitoring facility developed earlier was utilized to process and record the field data. In general, some thousands of well-defined events were recorded during the seven month study. Using suitable source location techniques AE/MS sources were found to be clustered along the actual longwall face; the majority lying within ± 60 ft of the face line. The techniques used in these studies are similarly applicable to monitoring the structural stability of highway tunnels during and following construction. A three-volume final report, available from NTIS, was prepared on these studies (Hardy and Beck, 1978; Hardy et al., 1977; Hardy et al., 1978). A brief paper on the study is available in the open literature (Hardy and Mowrey, 1976).

Rock Bolt Monitoring--In both mining and civil engineering, artificial support of rock structures is often provided by rock bolts. Such bolts are a major means of support in underground coal mines in the United States. A research study, aimed at developing instrumentation and associated techniques for the evaluation of bolt-rock interaction, has been carried out at Penn State (Unal et al., 1982; Unal and Hardy, 1984). During the study a special test facility was designed and experimental procedures developed in order to investigate the behavior of mechanical-type expansion-shell rock bolts under a variety of test conditions. High frequency AE/MS techniques were used here for the first time for the evaluation of rock bolt anchor stability. During the study a significant relationship was found between a number of the anchorage-testing parameters and the observed AE/MS activity. This result reinforces the possibility of developing an AE/MS based early warning system for monitoring the stability of bolted structures, such as mine or tunnel roofs and slope stabilizing structures.

Remote Monitoring and Prediction of Rock Drillability--At present a suitable method for on-line remote monitoring and prediction of rock drillability is not available. However, recent studies, on this topic have been carried out by Niitsuma and Chubachi (1986) at

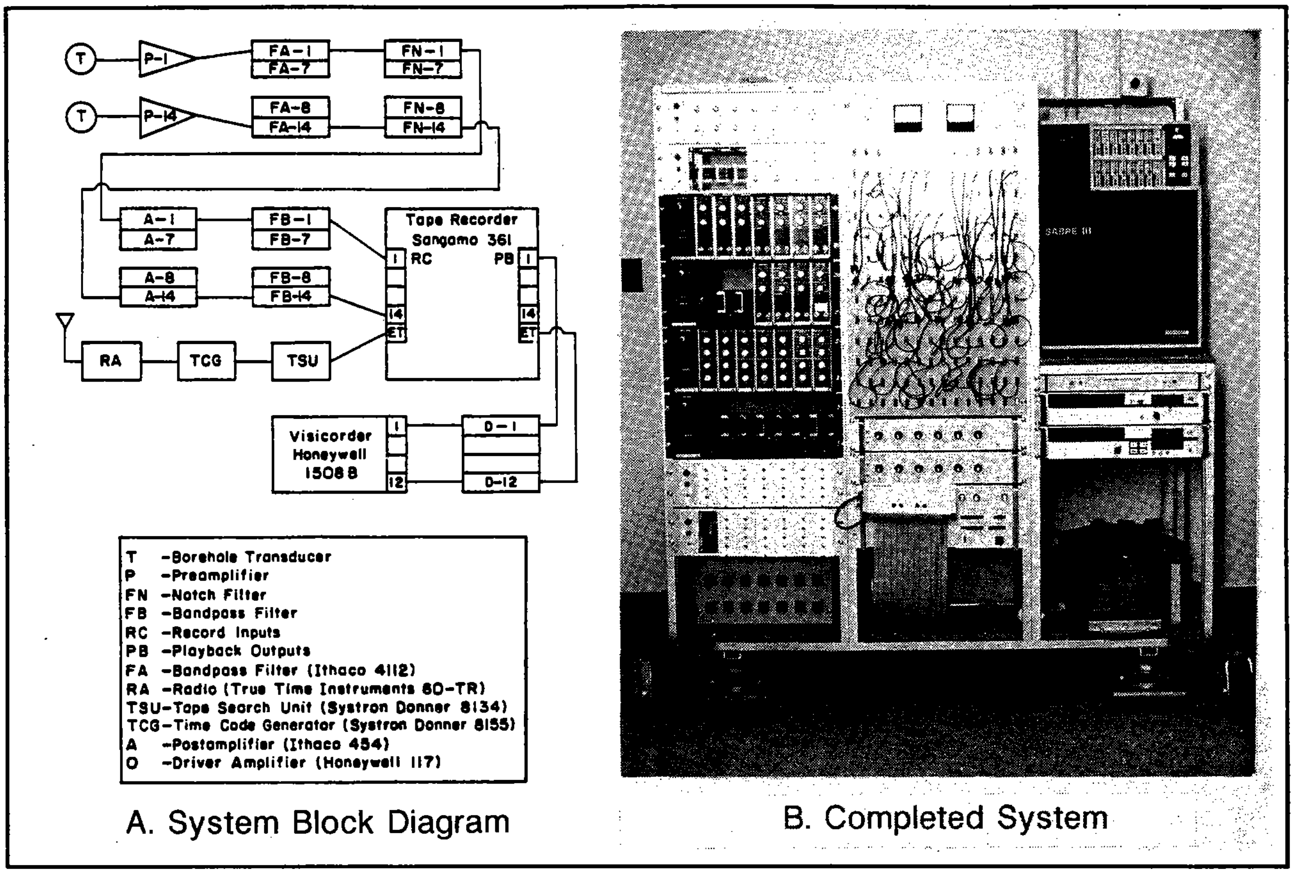

Figure 6: Mark II AE/MS monitoring system developed at Penn State University.

Tohoku University in Japan. These studies indicate that the basic characteristics of AE/MS data monitored during rock drilling appear to be dependent on a number of site specific factors such as the type of rock and its mechanical properties, and various operating factors such as drill bit type and condition, bit load and rate of advance. The studies indicate that the AE/MS technique has the potential of providing remote on-line evaluation of rock drilling operations. For example, AE/MS data collected during preliminary drilling from surface over a proposed excavation site, or ahead of a mine or tunnel face, could provide basic data on the "drillability" of the associated rocks and changes in rock characteristics with distance or position. Furthermore, during routine drilling operations, AE/MS data could provide continuous information on bit wear and drilling efficiency. Research relative to rock drillability was initiated at Penn State in early 1987. A recent paper describes the results of preliminary laboratory studies (Asanuma and Hardy, 1990).

Subsidence (Sinkhole) Studies--Recently The Penn State Rock Mechanics Laboratory carried out research, sponsored by the Federal Aviation Administration and The Pennsylvania Department of Transportation, relative to the location and stability evaluation of "sinkholes." The feasibility of a composite monitoring program involving both AE/MS and seismic tomography techniques was evaluated in a recent research study conducted at the Capital City Airport, near Harrisburg, Pennsylvania (Hardy, et al., 1986; Hardy and Belesky, 1987). The study indicated that the combination of these two techniques provides an effective means of locating shallow cavities and monitoring their stability. Further details of this study are presented later in this paper.

Effects of Blasting--AE/MS techniques appear to provide a practical means of evaluating the stability of natural and man-made structures during and after blasting. In this regard, studies have been underway by the writer since 1974 to evaluate the influence of underground and surface blasting on the stability of the strata overlying a shallow underground limestone mine north of Baltimore, Maryland. The study has involved the installation of an array of AE/MS transducers in and over the mine and the monitoring of AE/MS activity generated in the overlying strata prior to, during and following blasting. To date the effects of some 30 or more surface and underground blasts have been monitored at the Baltimore site. Although increases in local AE/MS activity were noted immediately following a number of these blasts, in all cases, the AE/MS rate dropped to the normal background level some hours later. Studies to date indicate that the stability of the underground mine and overlying strata are unaffected by the mine related blasting.

Mine and Tunnel Roof Stability--Unpredicted falls of rock from tunnel and mine roofs, both during development and subsequent use, continue to represent a major cause of fatalities and serious injuries. To date, no reliable, cost-effective means of predicting such roof falls has become commercially available. Studies are underway at Penn State to develop a practical, on-line, AE/MS system for continuous monitoring of mine and tunnel roof stability, location of potential areas of instability, and automatic advance warning of roof falls. Studies, relative to the use of a commercial high frequency AE/MS monitoring system to evaluate roof stability in underground coal mines (Ersavci, 1988), indicate that the system provides a well defined warning of such instabilities as roof falls, floor heave and pillar loading within a restricted monitoring zone. In order to monitor larger areas (e.g. long sections of tunnel or mine entry) studies are presently underway to develop mechanical waveguide systems which could be attached to existing roof bolts, perhaps over distances of many hundreds of feet (Hardy and Taioli, 1988; Hardy et al., 1989). With such waveguides, since the location of unstable zones will be accomplished using linear source location techniques, only two AE/MS transducers will be required, with possibly a third being utilized for automatic system calibration.

Slope Stability--Although AE/MS research in the field of slope stability, associated with both open pit mining and various civil engineering projects, has been relatively limited in the past, research interest in this area has increased in recent years (Hardy, 1989B). Recently the Penn State Rock Mechanics Laboratory carried out a study for the Pennsylvania Department of Transportation to evaluate the feasibility of using AE/MS techniques to evaluate the stability of highway rock slopes. A number of publications dealing with this study are available (Hardy, et al., 1988; Hardy and Kimble, 1990). This study will be described in more detail later in this paper.

3. Seismic Tomography

3.1 Basic concepts

In contract to the AE/MS method, seismic tomography utilizes external stimuli, i.e. seismic or mechanical signals, to evaluate the behavior of the structure under study. In its simplest form consider the use of a single transducer (T_1) and a single seismic source (S_1) as illustrated in Figure 7. In Figure 7A a void is located in the direct path between S_1 and T_1. Although a small amount of seismic energy may reach T_1 due to refraction and reflection, the majority of the energy is lost due to the presence of the void. Here the observed seismic signal will exhibit low velocity and high attenuation characteristics. In contrast, in Figure 7B there is a direct path between the source (S_2) and the transducer (T_2). As a result the observed seismic signal will exhibit high velocity and low attenuation characteristics.

In most field situations, an array of seismic transducers, often linear in form, is located in or on the structure, and a seismic source is activated at a series of selected positions. For each source location the associated seismic data is recorded at each transducer in the array.

After analysis this data provides the information necessary to construct a seismic parameter map covering the area bounded by the seismic transducers and the various seismic source locations. Seismic velocity is a commonly used mapping function, but seismic attenuation has also proven to be very useful in some applications. The cross-hole seismic technique illustrated in Figure 7, which is widely used in a variety of geotechnical applications, is a simplified form of seismic tomography.

In general, ST provides a means for developing a map of the distribution of a particular seismic parameter, for example, seismic-wave attenuation. The reconstruction algorithm necessary for analysis of seismic field data is similar to that developed initially for X-ray tomography in the medical field (Gordon, et al., 1970). Seismic tomography, in particular, differs from medical tomography in two ways, namely: the physical scale of the problem and the available scanning geometries. In the former, the physical dimensions for geophysical applications are much greater, requiring the use of lower signal frequencies for successful energy transmission. In the latter, medical tomography utilizes a multitude of scanning orientations with fixed source-to-receiver spacings, whereas geophysical tomography normally involves a limited number of scanning orientations with varying source-to-receiver spacings. A typical example of this is the tomographic imaging of the medium between two vertical boreholes (Lytle, et al., 1978). A number of papers dealing with the theory and application of seismic tomography are available in the literature. A recent paper by Sassa (1988) outlines a series of suggested methods for carrying out ST studies.

To date the major application of the ST technique by the writer has been in the location of shallow structural anomalies such as cavities (e.g. sinkholes). Research results indicate that such shallow structural anomalies markedly attenuate the amplitudes of waves that travel near the surface. To date, findings have shown that a cavity existing between a given seismic source point and receiver (transducer) will result in severe amplitude decay (Curro, 1983; Dresen, 1977; Greenfield et al., 1977). Collection of amplitude data associated with a number of source points and receiver positions would enable a plot to be made of the attenuation field. Such a plot could identify more precisely the location of shallow cavities.

3.2 Instrumentation

General--The purpose of ST instrumentation is to monitor the seismic signals received at a number of transducers as a result of a seismic disturbance at a single specified location (see Figure 8). In many ways the required instrumentation is similar to that used for AE/MS monitoring, which is described earlier in this paper. An additional transducer for "trigger" purposes is normally installed in the structure close to the seismic source or in the seismic source itself. The seismic signal provided by this source-transducer enables propagation velocity and other factors to be computed.

Seismic Sources--A variety of seismic sources are used in ST studies. These include "sparkers", air guns, explosive charges and simple mechanical impact. At Penn State limited studies have been carried out using a so-called "Buffalo Gun". Here shotgun shells, fired into water or mud filled boreholes, provide the desired seismic energy. The seismic signal from each type of source has a characteristic frequency response which determines its suitability for various applications.

Transducers--Accelerometers, velocity gages (geophones), and sometimes hyrophones are used as transducers in ST studies. As in the case of AE/MS measurements, the transducers must have the required sensitivity and frequency response. In some cases the transducers may be permanently installed on surface or in boreholes drilled specifically for that purpose. In other cases the tran#ducers may be temporarily clamped to the wall of a suitably located borehole, or in the case of hydrophone, hung at a specific depth in a water-filled borehole.

Transducer Location--The arrangement of seismic source location and transducer positions shown in Figure B has been used by the writer to successfully locate a number of shallow (0-5 ft depth) underground cavities. It consists of a line of shallow-burial, vertical-element geophones, and a parallel line of seismic source points which bound the desired study area. A similar instrumentation scheme was used for seismic evaluation in the Medford Cave study carried out by the U.S. Army Corps of Engineers (Curro, 1983). However, in this case, tomographic analysis of the associated data was not possible.

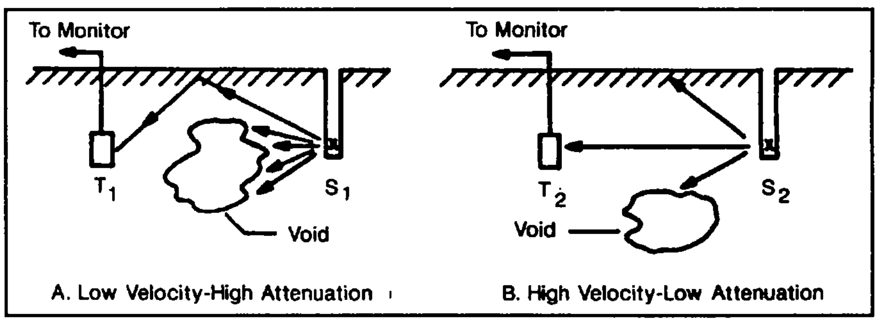

Figure 7: Effect of a void on seismic transmission during a "cross-hole" seismic test.

Monitoring Systems--Multi-channel monitoring systems similar to those used for AE/MS studies are commonly utilized to acquire data during ST studies. Each channel would have a configuration similar to that shown earlier in Figure 4. The Penn State Mark II AE/MS monitoring system, in which the complete data is stored on 14-channel magnetic tape, has been successfully used by the writer in a number of such field studies. Computer-based systems, incorporating high speed digitizers, have also

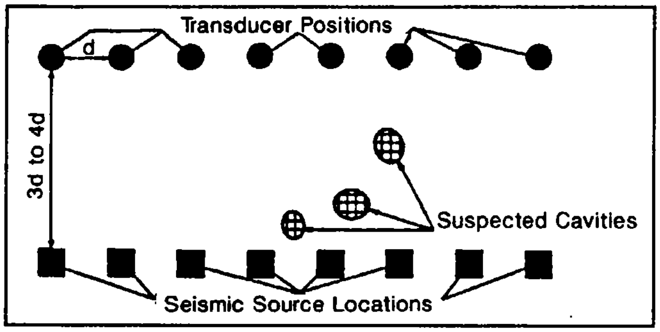

Figure 8: Typical Arrangement for Carrying Out a Seismic Tomography Study.

been extensively used.

3.3 Recent Applications

Seismic tomography has been applied with moderate success to various geophysical problems (Peterson, et al., 1985; Dines and Lytle, 1979; Lytle, et al., 1978; Inazaki and Takahashi, 1987; and Tweeton, 1988). Success of the technique is measured not only by the mathematical convergence and stability properties of the solution, but by the conformity of the image solution to the physical problem being studied. Limited angular coverage, curved raypaths, physical measurement error, or incomplete projections lead to "noise" within the completed projections.

Studies have shown that seismic waves traveling near the surface are sensitive to cavity features. In particular, research findings indicate that signal amplitudes attenuate significantly when the associated seismic waves pass through these structural anomalies. ST techniques are thus well suited to attenuation-parameter imaging, and signal-attenuation characteristics have provided useful information concerning rock structure in several geotechnical studies (Lytle, et al., 1978; Wong, et al., 1983). Consequently, application of surface-wave ST to cavity detection should improve the quality of cavity location. It is interesting to note, however, that until recently, seismic tomography has not been utilized for this purpose.

The recent Penn State geotechnical study carried out at Capital City Airport, near Harrisburg, Pennsylvania provided the opportunity to evaluate the feasibility of utilizing near-surface ST procedures for sinkholes location. Further details of this study are presented later in this paper.

4. Experimental Modal Analysis

4.1 Basic Concepts

Experimental modal analysis (EMA), like seismic tomography, also utilizes external stimuli. In this case, the source, often a hammer impact, is used to set the structure vibrating. Transducers located on the structure monitor the resulting vibration, and associated instrumentation collects the experimental data and determines a range of vibrational parameters. These parameters are dependent on the mechanical properties, geometry and defect character of the structure. In general the principles of EMA have been developed over a period of many years and have evolved from earlier techniques such as "Resonance Testing" and "Mechanical Impedance Methods". Clearly the availability of high speed digital computers has aided in the current rapid development of this subject.

Time does not permit a detailed discussion of the theory of modal analysis and modal testing and the recent book by D. J. Ewins (1986) is recommended for this purpose. In general the overall objective of such tests is to determine a set of modal properties for a structure. These properties consist of the associated natural frequencies, damping factors and mode shapes.

4.2 Instrumentation

Figure 9 illustrates a simple EMA system. In essence it involves an instrumented hammer which is used to impact the structure under study and cause it to vibrate. The hammer itself contains a transducer that provides a force versus time signal describing the character of the actual hammer impact. This signal is feed to a signal conditioning facilty ("power unit") and then to one channel of a suitable recorder (AT&T PC6300). A transducer (accelerometer) attached to the test structure monitors the vibration of the structure due to the hammer impact. The output of the transducer is fed, via a second signal conditioning facility ("power unit"), to a second recorder channel.

Two basically different types of EMA techniques are utilized, namely "roving excitation" or "fixed excitation". In the former, a single transducer is attached to the structure and the instrumented hammer is used to impact the structure at a number of selected points. The latter technique involves a fixed source of excitation (e.g. hammer impact at a single point) and a series of transducers attached to the structure at a number of selected points. There are obvious advantage and disadvantages of both EMA techniques.

Penn State studies to date have utilized the roving excitation technique. Here only one transducer and associated signal conditioning and recording channel are required. The technique, however, is labor intensive since even a small test structure may involve 25-30 impact points. Furthermore, 10-20 individual impacts may be required at each impact point for "stacking" and improvement in the signal-to-noise ratio. Such studies therefore necessitate 500 or more separate manual impacts.

Optimizing the EMA system for a specific application requires "electing a hammer with suitable force capacity and tip hardness, a transducer with suitable frequency range and sensitivity, and a computer-based recording system with sufficient digitization speed and storage capacity. EMA data is usually collected and processed by some form of commercial data acquisition and processing software resident in an associated computer facility.

4.3 Recent Applications

To date experimental modal analysis has not been applied extensively to geotechnical structures. An early paper by Davis and Dunn (1974) evaluated the use of EMA to determine the length and continuity of in-place piles, and for the evaluation of the quality of the associated concrete and pile-ground anchorage quality. Olson and Wright (1989) have used a simplified form of EMA, the "Impulse Response Method", to study debonding in concrete, and to evaluate the quality of subgrade support and to detect the presence of voids. Carino and Sansalone (1989) have used another simplified form of EMA, the "Impact-Echo Method" to study delamination in concrete bridge decks. Olson and Wright (1989) have also utilized the impact-echo technique to evaluate the integrity of a concrete box girder bridge. Detailed EMA studies in the geotechnical area are very limited; however, a recent paper by Farrar (1989) describes such studies carried out on a shear-wall element and a model developed for the investigation of soil-structure interaction.

In the Penn State Rock Mechanics Laboratory only a limited number of EMA studies have been undertaken to date. These include a basic laboratory study carried out on a large rock plate (Sun and Hardy, 1990) and two field studies on a highway rock slope. Further details of the rock slope studies will be presented later in this paper.

CURRENT PENN STATE APPLICATIONS

In the following sections, further details on a number of current Penn State geotechnical field projects will be presented. These will provide a more detailed appreciation of the field application of the various NDT techniques described earlier in this paper.

1. Slope Stability Monitoring

1.1 Introduction

The design of stable rock and soil slopes and the stabilization of landslide areas is one of the most important problems in the field of geotechnical engineering today. Civil engineering applications include dams, highway road cuts, railway cuts, and tunnel portals. Slope stability is also a primary economic consideration in the operation of all types of surface mines. For many years, conventional surveying and other displacement monitoring techniques have been used to evaluate the stability of rock and soil slopes and landslide areas. More recently, an increasing number of studies have been reported in which acoustic emission/microseismic (AE/MS) techniques have been successfully utilized for such purposes. A recent State-of-the-Art Review by the writer (Hardy, 1989A) considers the historical development of the AE/MS technique as applied to slope and landslide monitoring, and briefly discusses a number of the successful applications.

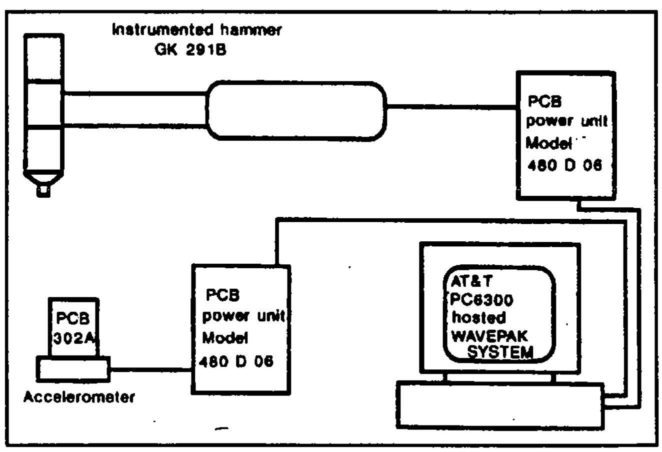

Figure 9: Typical System for Conducting Experimental Modal Analysis Studies.

In 1985 a project, supported by the Pennsylvania and U.S. Departments of Transportation, to investigate the use of AE/MS techniques to monitor highway rock slopes was initiated at the Pennsylvania State University. Although some consideration was given to past studies associated with soft ground slopes, the experimental phase of the project was limited to rock slopes, and specifically rock slopes associated with highway construction. In 1989 the application of EMA techniques to slope stability evaluation was first considered by the writer, and later that year an associated field study was initiated.

1.2 Field Sites

Studies have been carried out at the "Lamar" and "Wilkes-Barre" project field sites. These were located on interstate highways I-81 and I-80, in north-eastern and central Pennsylvania, respectively. A detailed description and location of these sites are given elsewhere (Hardy, et al., 1988). Figure 10 illustrates the common types of slope instability located at these sites. In all three types of slope failure, fractures must be initiated and grow to euch an extent that a unit of rock becomes mechanically ieolated from the rock mass. Relative motion then occurs, the isolated unit slides free of the rock mass, and falls, bounces, or slides to the bottom of the slope.

1.3 AE/MS Studies

Field studies involving the application of both low frequency (LF) and high frequency (HF) AE/MS monitoring techniques were carried out at the Lamar and Wilkes-Barre field sites. LF studies utilized conventional geophones and in some cases accelerometers. Geophones were installed in shallow holes in the berm at both field site areas and in deeper holes on the slope crest at the Lamar (LA-1) site. Accelerometers were cemented to steel plates, bolted and epoxyed to the slope face. LF multi-channel AE/MS data (0-1000 Hz) was processed and recorded on magnetic tape using the Penn State Mark II mobile microseismic monitoring facility described earlier.

High frequency (HF) studies utilized conventional resonant-type accelerometers (AE-Transducers), having a resonant frequency of 30 kHz and a useful range of approximately 15kHz - 45kHz. Limited HF studies utilizing transducers having a resonant frequency of 175 kHz were also carried out. In all cases AE-transducers were attached to metal plates, previously bolted and epoxyed to the slope face, using commercial magnetic mounts. A suitable preamplifier interfaced the HF transducer to a commercial single channel, battery operated monitoring system (AET model 240GR).

Using the LF and HF AE/MS monitoring facilities just described a series of studies were carried out at the Lamar and Wilkes-Barre test areas. These included the evaluation of a variety of AE/MS transducer installation techniques; investigation of the effect of highway traffic on AE/MS system optimization; evaluation of overall monitoring system sensitivity using various calibration sources (e.g. impact mass, sliding mass, explosive devices, etc.); estimation of source location accuracy; and development of an automated rock fall monitor system. A recent paper (Hardy and Kimble, 1990) outlines these studies in more detail. In general the slopes at the two test areas were innately stable. In order to evaluate the feasibility of using AE/MS techniques to monitoring highway rock slope stability, a variety of artificial sources were utilized to generate "AE/MS-like" activity. Artificial sources included acoustic sources (e.g. noise from

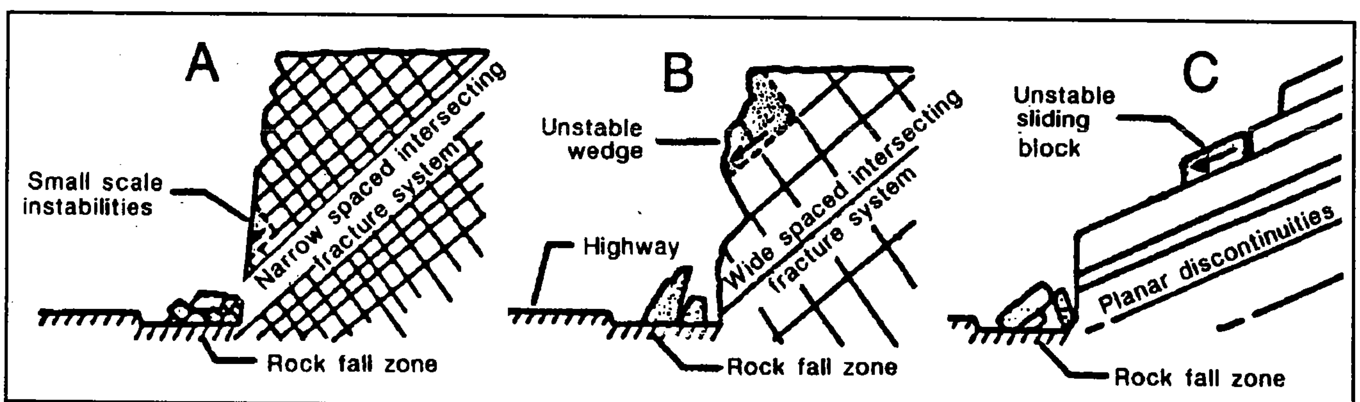

Figure 10: Three Typical Types of Unstable Highway Rock Slopes Associated with the Penn State Slope Stability Field Studies.

breakage of thin wood strip or dry reed), highway traffic, down-hole explosive charges, surface shear and impact, and small scale rock failure. Figure 11 shows a number of systems for producing impact and shear sources. A modification of the shear system shown in Figure llC was also utilized to generate the effect of surface shear on horizontal surfaces.

Due to space limitations only a few of the major results of the LF AE/MS study will be included here. Further details of these, and the associated HF studies are presented in a recent PennDOT report (Hardy, et al., 1988).

(1) Cutural Noise - The major cultural noise at both the Lamar and Wilkes-Barre sites was due to highway traffic. Signal amplitudes varied from 246 in./sec for a passenger car to 16,000 in./sec for a heavily loaded transport truck. The major frequency components were in the 7-110 Hz range at both sites. At the Lamar LA-l site a few well defined higher frequency components were observed at 197, 324 and 450 Hz. Since simulated AE/MS events (impact, shear, etc.) were found to exhibit frequencies in the range 6-90 Hz, meaningful LF AE/MS studies of slope stability, must be carried out at times when vehicular traffic is at a minimum. Alternatively, suitable monitoring systems incorporating selective or adaptive filtering, pattern recognition, etc. must be developed to electronically eliminate the effects of vehicular traffic.

(2) Attenuation - The attenuation studies on the slope area at the Lamar LA-l site indicated that a LF transducer array located in the slope crest area could detect simulated AE/MS events near the slope toe, approximately 45 ft below the slope crest. In general, however, the attenuation characteristics of the slope area were highly complex, depending to a great extent on the fracture distribution within the overall slope structure. Studies along the berm at the Lamar LA-l site provided anomalous, results suggesting a marked decrease in attenuation characteristics for distances greater than 40 ft from the seismic source. The observed effect is assumed to be due to the structural complexity of this region.

(3) Sliding Block Studies - As a means of simulating rock slope failure a series of rock block sliding studies were undertaken using the system shown earlier in Figure llC. Tests were carried out on both the face and crest of the slope. During crest studies, for propagation distances in the range 10 to 37 ft, the frequency spectra of signals from block sliding ranged from 6 Hz - 178 Hz. In general, however, the major peak in the frequency spectra was in the 22 Hz - 40 Hz range. For similar propagation distances, the observed maximum particle velocities were found to range from 33×10^{-6} - 190×10^{-6} in./sec. The ability to detect such low level signals, however, depends on the ambient background level and the associated signal-to-noise ratio (SNR). Slope face studies suggested that the use of suitable signal processing techniques should allow routine detection of LF AE/MS signals due to "sliding" for distances of 100 ft or better in a highly fractured structure such as the LA-l site slope. A larger range of detection would be expected in a slope with a lower level of fracturing.

(4) Velocity Model - The ability to accurately locate

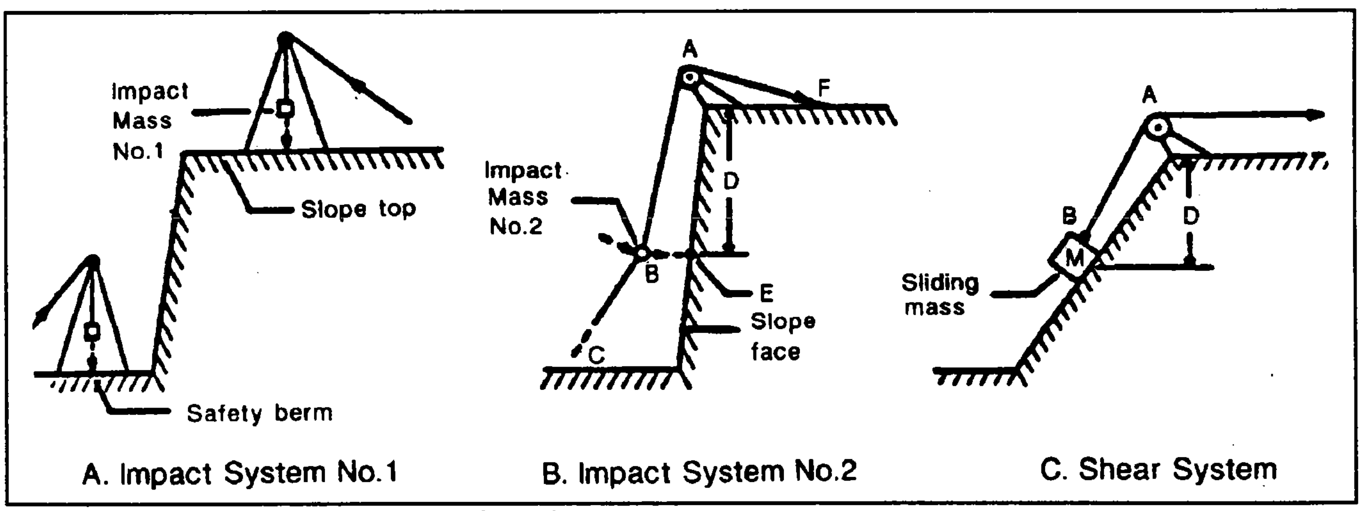

Figure 11: Systems Producing Impact and Shear Sources.

AE/MS sources necessitates a well defined velocity model for the structure under study. At the Lamar LA-l site the impact systems shown earlier in Figure llA, and the permanently installed transducer arrays located in the slope crest and berm areas were used to evaluate velocity models for the structure. For example, the velocity data obtained for the slope crest area suggests a reasonably isotropic velocity model. At this site, P-wave velocities obtained during one study were found to range from 2641 - 3792 ft/sec with an average value of 3180 ft/sec. These are some 300% higher than the P-wave velocities determined in the laboratory for intact core obtained from the slope crest area.

(5) Source Location - Source location studies were carried out at the Lamar La-l site using an artificial source (see Figure llB). These studies indicated that at positions near the center of the slope (15 - 35 ft from the slope crest) source location errors were in the range of 15 ft, with values decreasing to less than 5 ft at the center of the slope. Considering the relatively simplified analysis used, the results are extremely encouraging. To the writer's knowledge this is the first reported case where meaningful AE/MS source location has been achieved for a highway rock slope.

Another important component of the project, was the development and testing of a prototype rock fall monitor (RFM). With additional development this RFM should prove useful, both as a means of recording rock fall patterns for specific slopes and warning motorists of the current rock fall danger, and also as a means of providing data which may be useful in predicting subsequent major slope instabilities.

To date, the results of studies using AE/MS techniques to monitor slope stability have been most encouraging, and further AE/MS studies are planned in the future.

1.4 Modal Analysis Studies

As a result of a number of technical problems in applying AE/MS techniques to highway slope monitoring, studies are also presently underway to investigate the feasibility of using Experimental Modal Analysis (EMA) in those cases where highway traffic and other cultural noise are excessive. As described earlier in this paper, EMA utilizes an artificial mechanical source (e.g. impact) to cause the test structure to vibrate. The analysis of the vibrational characteristics of the structure provide information on its composition and degree of mechanical stability.

A planar slope, located at the Wilkes-Barre I-81 field site was selected for the EMA studies. A preliminary study was carried out at this site during the summer of 1989

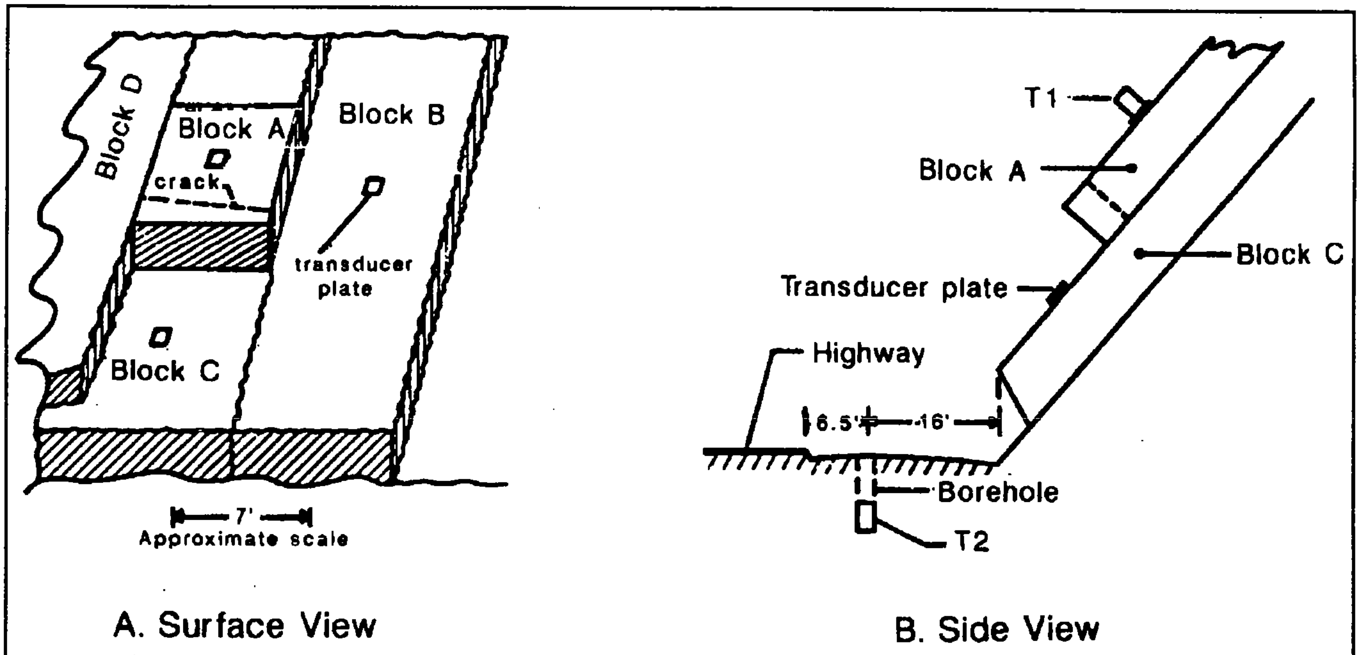

Figure 12: Location of Test Block A, relative to other slope components at Wilkes-Barre Test Site. (T1 & T2 are locations of monitoring transducers.)

(X. Sun, 1990) and the analysis of data from a second, more detailed, study at that site is presently underway. Figure 12 shows the location of test block A relative to other slope components.

During one stage of the test program a low frequency accelerometer (TI) was attached to block A and the block was mechanically impacted at a number of selected points, using a specially instrumented hammer. The impact signal from the hammer and the transducer (TI) output were recorded using the facilities shown in Figure 13. Analysis of the data collected during impact provided a number of parameters, including the fundamental mode shapes of vibration of the block. A series of block modifications were then carried out which involved the sequential development of a pre-existing crack (see Figure 12A). At various etages of crack development a detailed modal analysis study was curved out on the block. Figure 14 illustrates the mode shape for the first vibrational mode (67-82 Hz) prior to block modification and for three subsequent stages of crack development. The differences in mode shape are clearly evident. Detailed analysis of the experimental data is underway and a final report on the modal analysis feasibility study will be completed later in 1991. Results to date suggest that EMA may be a feasible method of monitoring stability in planar rock slopes and possibly in other types of rock structures.

2. Sinkhole Monitoring

2.1 Introduction

Sinkholes are a common feature in karst terrain, i.e. terrain produced by the solution of carbonate rocks such as limestone and dolomite. The process involves the solutioning of the susceptible rock material and the subsequent downward migration of eroded unconsolidated deposits that overlie the irregular or pinnacled bedrock. A damaging result of this process is the formation of cavities near the ground surface. The driving mechanism behind this activity is some change in either the flow of water drainage or the level of the natural water table.

The Capital City Airport, located near Harrisburg, Pennsylvania, has experienced a large number of sinkhole occurrences throughout its existence. Figure 15 shows a section of one runway, prior to remedial grouting and resurfacing, illustrating the effects of extensive sinkhole activity. Geotechnical investigations have confirmed the presence of highly eroded, cavity-ridden strata beneath the airport property. In the past, several engineering alternatives were proposed ranging from relocation of the facility to major reinforcement of the underlying strata and subsequent replacement of the runway pavement. Due to social, economic and political factors, the selected

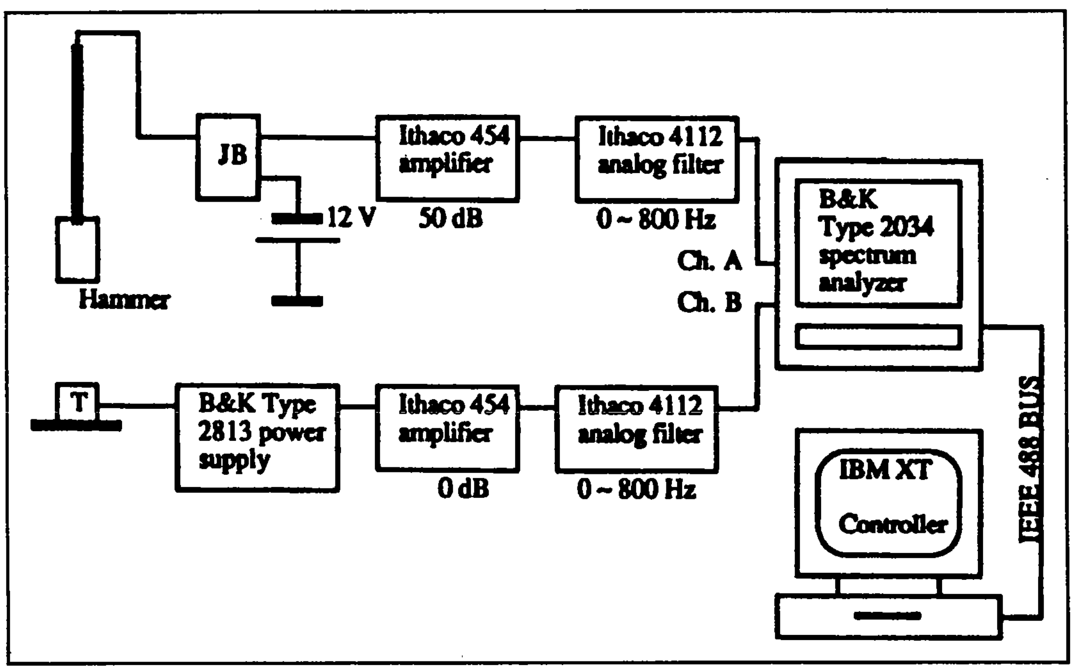

Figure 13: Block Diagram of Instrumentation Setup for Field Modal Testing. (JB-junction box, T-transducer.)

alternative involved the remedial grouting of open cavities and pavement resurfacing.

Concurrent with this remedial effort, a monitoring technique was sought to minimize the risk imposed by pavement collapse due to cavity instabilities. The feasibility of a composite monitoring program involving acoustic emission/microseismic (AE/MS) and seismic tomography (ST) techniques was evaluated in a recent research study conducted at the airport. Based on the results obtained to date, the AE/MS technique, utilizing zonal location and hit-sequence location, was shown to be capable of accurate source location in such an environment. ST, utilizing signal amplitude data, proved to be sensitive to cavity-prone regions. The combination of these two techniques provides a means of locating cavities and monitoring their stability. Such a composite technique integrates well into remedial engineering solutions by identifying areas needing immediate attention and allowing subsequent monitoring of the effectiveness of the implemented remedial measures.

2.2 Field Site

One phase of a major Penn DOT Study prior to 1984 (see Hardy et al., 1986) involved the drilling of a total of 26 boreholes to correlate with the results of earlier resistivity and fracture-trace studies. Rock was encountered initially at depths of from 4 to 50 ft and the borehole logs were in general, erratic. Open cavities, mud-filled cavities, and zones of low drill-penetration resistance and "no air return" were encountered frequently. Soil cavities with dimensions in excess of 20 ft were intercepted along portions of the active runways. On the average, 18 percent of the total drilling length encountered open or soil-filled voids. Such media severely complicate the application of indirect measurement techniques such as AE/MS and ST.

Two specific sites were selected for research studies. Test area No.1 was an approximately elliptical area (350' x 700') which included one of the most seriously affected areas of Runway 8-26. Test area No. 2 was a rectangular grassy area (100' x 200') located adjacent to Taxiway A. Holes for transducer installation up to 20 ft in depth, drilled at the two test areas indicated that to depths of at

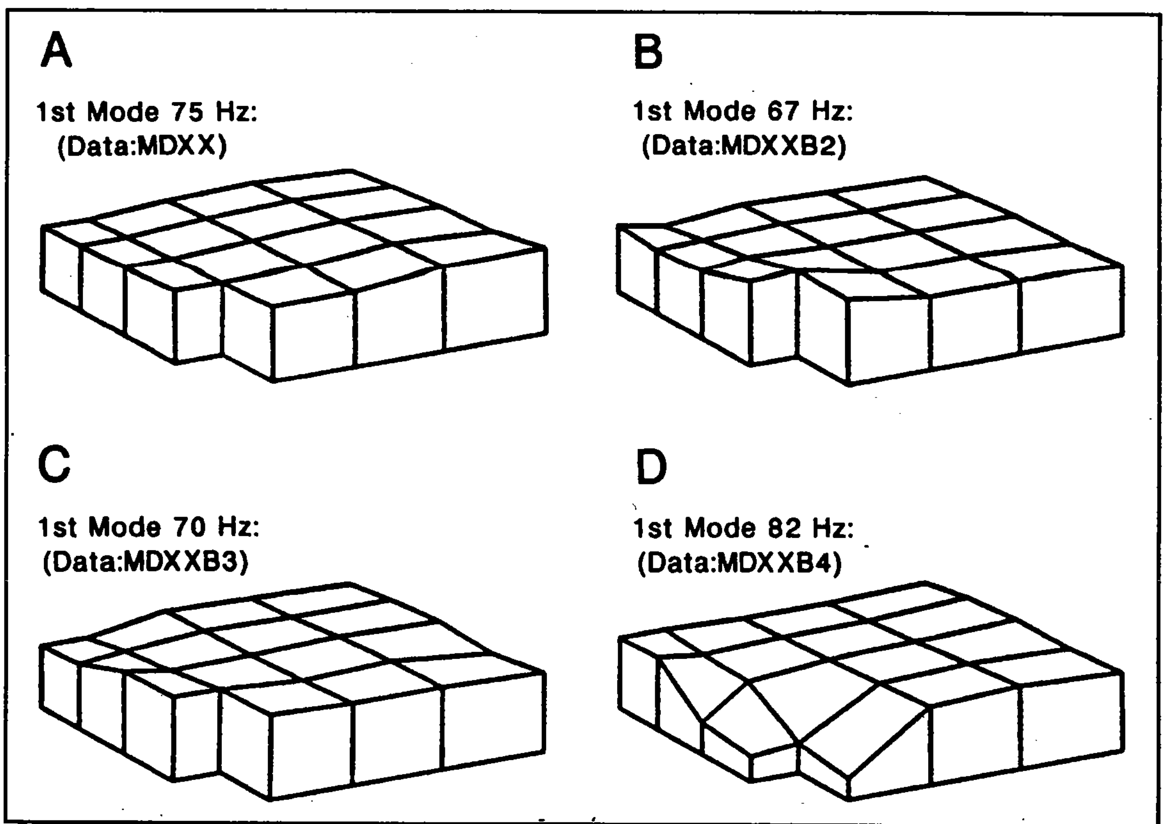

Figure 14: Mode Shapes for Block A at Various States of Block Modification.

Figure 15: Section of Airport Runway Showing Effects of Sinkhole Activity.

least 20 ft the subsurface was extremely complex, consisting of regions of soil, weathered dolomite and competent dolomite. Often layers of soil and weathered dolomite were found to lie below layers of competent dolomite. A variety of small and large voids were intercepted during drilling. Transducer holes at test area No.2 were approximately 3 ft deep and at this depth appeared to be located in soil. One hole at this test area encountered a void at a depth of approximately 2 ft below surface.

2.2 AE/MS Studies

Typical geophones, mounted in marsh-type cases, were used as transducers at all test areas. At a number of locations transducers were permanently installed at depths of 10, 20 and 30 ft. During selected studies, additional transducers were temporarily installed in shallow holes up to 3 ft in depth. AE/MS activity was monitored and recorded using the Mark II mobile monitoring facility facility described earlier in the paper. For most studies filter setting were established to pass signals in the range 5-100 Hz with notch filters at 60, 120, 180 and 540 Hz to suppress the power line fundamental and selected harmonics.

A range of AE/MS studies were carried out at the airport site to investigate the following:

(1) time-series and frequency spectra of typical background and cultural activities common to an airport site.(e.g. aircraft taxiing, taking-off and landing; nearby trains and automobile traffic; etc.).

(2) attenuation characteristics of the soil and sub-soil areas.

(3) source location accuracy using a variety of location techniques.

The results of these studies are presented in detail elsewhere (Hardy, et al., 1986) and provide a wealth of information applicable to the design of AE/MS monitoring systems for use at airport sites. Unfortunately runway repairs (pressure grouting) was undertaken before studies could be initiated to monitor AE/MS activity of major sinkholes in test area No. 1. In addition weather conditions generally became unfavorable for the initiation and development of major new sinkholes in test area No. 2.

One of the most important findings of the study was relative to the study of AE/MS source location accuracy. In general, AE/MS event location is more difficult in near-surface applications due to a number of geotechnical complexities, including the following:

(1) Relatively high signal attenuation, which leads to ambiguities in identifying and evaluating the associated seismic-wave arrivals and effectively limits the distance over which usable signals can be detected.

(2) Highly complex velocity models which result in potentially large deviations between actual seismic velocities and the estimated velocities utilized in source-location analysis.

As a result, in near-surface applications, conventional least-squares based AE/MS source-location procedures, which are normally employed to provide precise location coordinates, far exceed the levels of precision of such input parameters as seismic wave arrival times and velocity-model characteristics. In the current study, the conventional source-location approach as well as a new approach, referred to as zonal location, were investigated.

According to Fowler (1984) zonal-location techniques offer four advantages over conventional, least-squares based source-location routines, namely: (a) severe attenuation is permissible; (b) accurate velocity models are not strictly necessary; (c) computation time is reduced considerably and zonal location can often be performed in real time; and (d) zonal-location algorithms make use of the majority of incoming data. Although several variations exist, zonal-location algorithms simply search for the transducer with the first signal arrival (hit) and immediately establish a primary zone of activity. Successive wave arrivals at additional transducer sites and, in some algorithms the arrival-time differences, are used to refine the zone of activity. The use of the arrival-time sequence and arrival-time differences to obtain a refined source location is referred to by some authors as hit-sequence location (HSL). Es#entially the HSL method defines a smaller subzone within the original primary zone from which the event originated as illustrated in Figure 16.

Data collected during this study allowed a comparison to be made of the source-location accuracy associated with the conventional least-squares, zonal and hit-sequence techniques. In order to provide suitable test data a seismic source was activated at a number of accurately known positions throughout the AE/MS array installed earlier at test area No. 2. Twenty-eight calibration-source positions within the array (CS points) and ten calibration-source positions outside of the array (OS points) were utilized. Arrival-time-difference data and the ordered sequence of transducers to receive a given signal were compiled for each calibration-source point. In general, based on conventional least-squares source-location methods, an error of 5 ft in a computed source location was considered acceptable. Relative to the zonal and hit-sequence techniques this corresponds to the true source being located in an area of approximately 80 ft centered on the calculated source location. Calculated sources unable to satisfy the preceding criteria were considered to be unacceptable.

Generally the zonal and HSL techniques were found to be the most suitable. Figure 17 indicates the location of the seven transducers (SCl-SC7) in the AE/MS monitoring array at test area No. 2, and the results for two of the calibration tests. In each case, the solid dot is the true position of the activity and the open circle is the conventionally located source position. In case A, zonal location selects the area immediately surrounding transducer position SC1 (bold outline). Two-hit based hit-sequence location (HSL) yields the crosshatched area representing the hit sequence SCl followed by SC6. The three-hit HSL yields the shaded zone representing the hit sequence of SCl, SC6, and SC7. In this case, zonal and HSL results coincide exactly with the known source position. In case B, conventional location yields very poor results but zonal and two-hit HSL results are excellent.

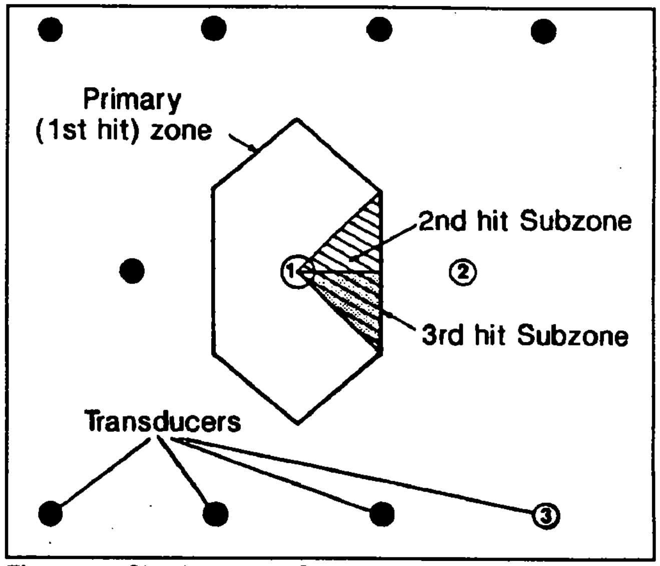

Figure 16: Simplified Field Situation Illustrating the Zonal and Hit-Sequence Concept.

2.4 Seismic Tomography Studies

Although not originally part of the planned airport study, preliminary ST studies carried out at test area No. 2 were found to provide encouraging results. During initial studies a rectangular test area approximately 120 ft x 35 ft, was selected. As illustrated in Figure 18 a series of 12 source holes (SP-1 to SP-12) were established along one side of the test area, and a series of 11 geophone transducers (AS-1 to AS-11) were installed along the opposite side. During testing the transducer outputs were monitored by the Penn State Mark II AE/MS monitoring facility.

Mechanical impact by a hammer blow to a rod and attached plate, located at the bottom of the hole, provided the seismic source. This impact was applied two or three times at each of the 12 source holes in succession and geophone data recorded. This data was subsequently analyzed to provide both velocity and attenuation data for each source-transducer path, for a total of 132 paths. A tomography algorithm, based on the algebraic reconstruction technique (ART), developed at the University of California - Berkeley was used to analyze the field data. The algorithm is described in the paper by Peterson et al. (1985).

Although both velocity- and attenuation-based analysis have been carried out, the latter appears most suitable for near-surface cavity locations. For example, Figure 18 illustrates the attenuation-based tomographic image for an area containing open sinkholes and surface depressions. The darkest grid elements correspond to high signal attenuation. The points ASl, . . ., ASll refer to the eleven transducer positions and SPl, . . ., SP12 refer to the twelve seismic source points (hammer impact points) used in this study. A 10 ft spacing was used between successive transducer and source points. Each grid element is approximately 8 ft wide.

The image in Figure 18 contains four areas of high signal attenuation (darkest grid elements). The largest area near the center of the image corresponds to an area containing existing open sinkholes. The dark element located to the right of transducer ASl corresponds precisely with a void located during transducer installation. The high-attenuation region near ASlO and ASll coincides with an area where auger drilling met with little resistance. The slightly darker region to the left of source point SP12 corresponds to the location of several obvious surface depressions.

To confirm the replicability of the method, two separate seismic surveys were conducted over the same region. The resulting images were essentially identical. In addition, to evaluate the reliability of the technique for cavity detection, a seismic survey was conducted over a region devoid of surface features directly attributed to sinkhole development. The resulting tomographic image contained little evidence of high signal attenuation.

Although the ST studies carried out at the Capital City Airport site were of a relatively limited nature they clearly showed the usefulness of this technique to determine the location of shallow sinkholes. Furthermore it is clear that a combination of ST and AE/MS techniques provide a useful approach for the

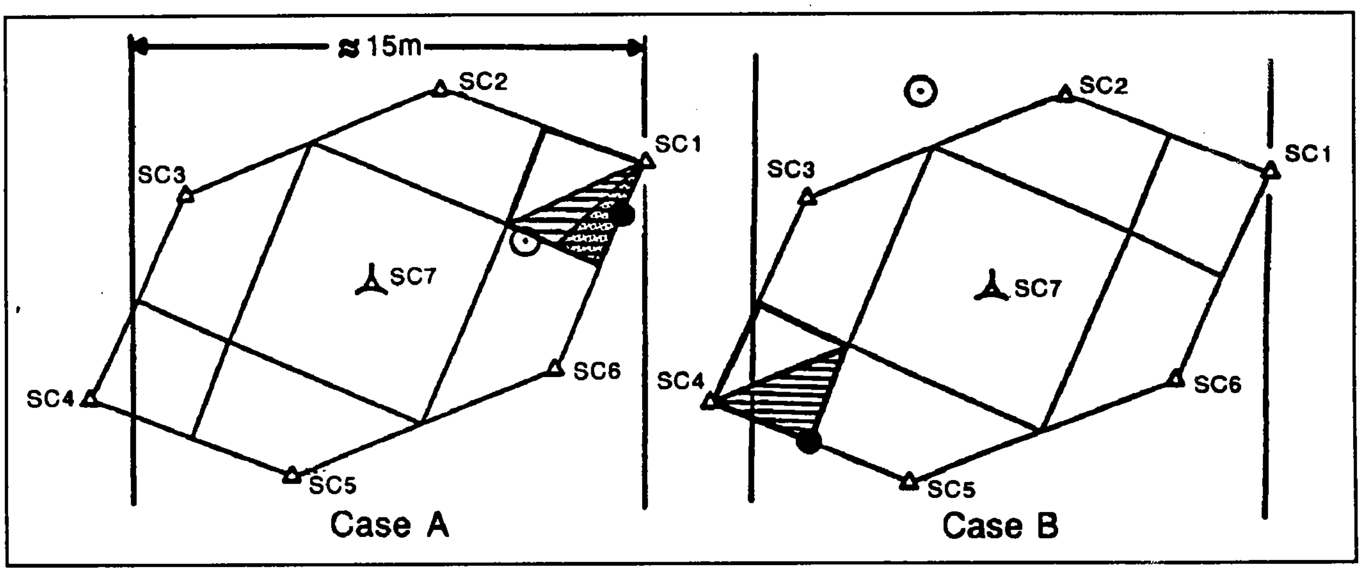

Figure:17
Comparison of computed source locations using conventional, zonal, and hit-sequence techniques. (Triangles indicate transducer positions, solid and open dots indicate true source and conventionally computed source locations. Bold outlined regions indicate zonal location. Hatched and hatched-stippled regions indicate two- and three-hit hit sequence locations. 1m = 3.281 ft.)

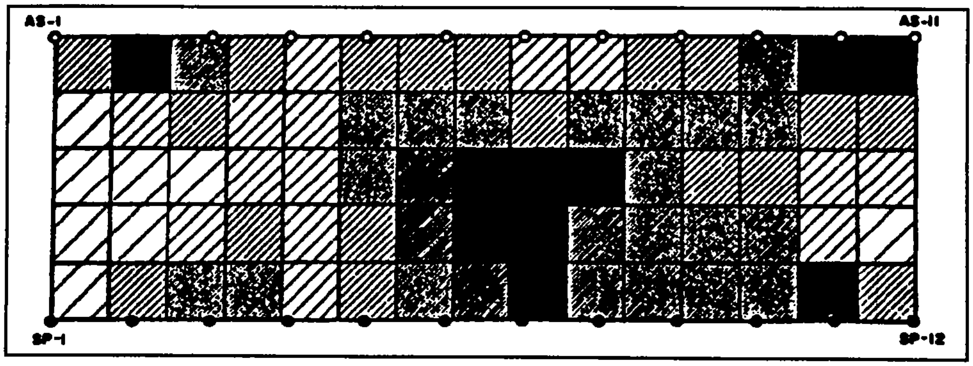

Figure:18
Attenuation-based tomographic image of original survey region at test area No. 2 using October 1, 1985 data. (Attenuation increases with degree of shading of grid elements. Width and length of survey region are approximately 35 and 120 ft. repectively.)

location and stability monitoring of shallow sinkholes.

CONCLUSIONS

In this paper the writer has described three non-destructive testing techniques which have wide application in the geotechnical field. These techniques, based on acoustic emission/microseismic activity, seismic tomography and experimental modal analysis, provide unique methods for evaluating the defect character and mechanical stability of rock and soil structures. Each technique has certain limitations and further studies are required to optimize their usefulness and ease of application in specific field situations. Clearly, however, effort expended in learning to better understand these techniques and in applying them to various geotechnical problems will be most rewarding. Those involved in geotechnical engineering are encouraged to learn more about available NDT methods and where possible to utilize them for routine evaluation of structural stability and the study of more complex geotechnical problems.

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ROCK SLOPE EXCAVATION AND STABILIZATION METHODS IN HIGHWAY CONSTRUCTION: INTERSTATE 287 EXTENSION, NEW JERSEY

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ABSTRACT

Construction of a 20 mile extension of Interstate 287 in northern New Jersey was started in 1986 and is expected to be completed in 1994. Construction includes approximately 9 miles of rock cuts up to 220 feet high and excavation of approximately 11 million cubic yards of rock.

The alignment follows and crosses the Ramapo Fault which has had a complex tectonic history and is the major structural component of the region. Design and construction of rock slopes in such highly variable structural and geologic terrain pose many interesting problems to the engineering geologist.

The specifications for stabilization measures including tensioned rockbolts, rock dowels, tendons, shotcrete, and wire mesh were reviewed and revised to ensure the most up-to-date techniques were being used. The specifications for blasting, excavation, and scaling of the finished cut face were modified to enable access for inspection of slope conditions. All of these specification changes were implemented in order to allow for thorough evaluation of the varying geologic conditions and to minimize impact to construction progress while stabilization measures are being installed. This paper describes examples of slope stability problems, stabilization methods, and specification language related to rock slope excavation for this project.

PROJECT OVERVIEW

The last section of Interstate 287 in northern New Jersey is presently under construction. The long awaited, and badly needed section of road extends 20 miles from Montville, New Jersey to Suffern, New York where it will tie in with the New York State Thruway (Figure 1). At the time of the preparation of this paper, excavation of 80 to 90 percent of the estimated 11 million cubic yards of rock has been completed. Cost for the project has been estimated at more than $600 million.

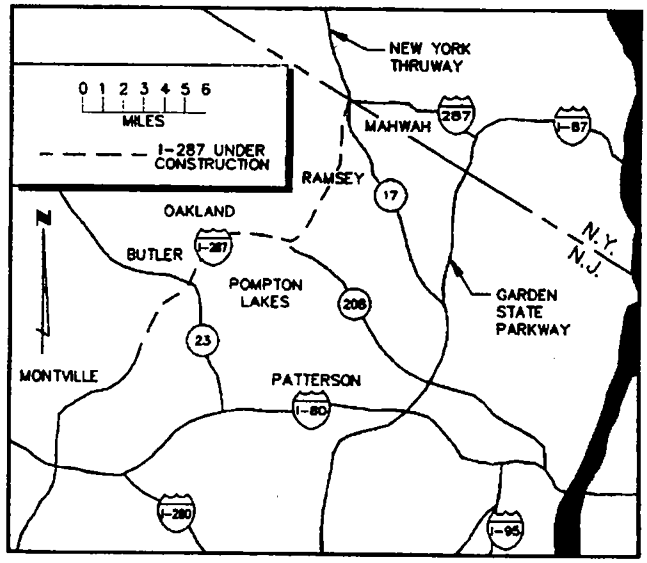

Figure 1: Project Location

Original geologic investigations for rock slope design recommendations were performed from 1982 to 1983. Investigation along the southern six miles of the project was conducted by Woodward Clyde Consultants; the remaining 14 mile section was conducted by Golder Associates Inc. After construction began in 1986, Golder Associates Inc. was retained by the New Jersey Department of Transportation (NJDOT) to assist in supervision of rock slope excavation and to provide on-site design recommendations for slope stabilization and protection.

A project of this scope presents a variety of challenges from both a construction and engineering geologic perspective. Rock excavation has been conducted by four separate contractors along the 20 miles of roadway. They have shown a varying familiarity with rock slope stabilization techniques. It has been the objective of the engineering geology staff to evaluate and mitigate adverse rock slope conditions as excavation proceeds while trying to minimize impact to construction/excavation sequences. In most cases, for smaller localized instabilities this has been achieved. There have been situations where identification of potential problems could not be made concurrently with excavation. This has happened when structural and/or lithologic changes were identified in lower portions of cuts that had the potential to destabilize the rock above. The situation described has presented the engineering geology staff with a difficult choice of 1) possibly incurring unnecessary expense and inappropriate solutions by stabilization recommendations that are based on insufficient information, or 2) to allow construction to continue in the desire to obtain more information, but possibly risking a slope failure before stabilization measures are implemented. This paper will discuss some of the geologic conditions encountered in the area and the methods implemented to produce stable rock slopes.

GENERAL GEOLOGY

Rock excavation consists of both through and side hill cuts along the flanks of the Ramapo Mountains, which form a series of low ranges of Precambian Age Gneissic rocks immediately west of the Ramapo Border Fault. The Ramapo Mountains fall within the New Jersey Highlands section of the Appalachian Mountains physiographic province and are separated from Jurassic and Triassic sediments and lava flows of the Newark Basin to the east by a series of parallel faults known as the Ramapo Fault system (Reference 1). The Ramapo Fault is the major structural component of the region and follows a general N40°E trend and dips steeply to the southeast. The grain of the rock within the slopes also follows this same general strike and dip. Rock types are primarily Gneisses and Amphibolites with varying dominant mineral assemblages.

The tectonic history of the Ramapo Fault and adjacent rock of the Ramapo Mountains is very complex. Movement along the fault initially developed during the Grenville Orogeny, approximately 1.1 billion years ago. Subsequent movement has occurred during the Paleozoic, Mesozoic, and possibly Tertiary and Recent Ages (Reference 2). The sense of movement along the fault throughout its history has, at the very least, included right lateral oblique, thrust, dip-slip, and strike slip behavior.

All of the rock along the alignment was subjected to ice loading and erosion during the last period of continental glaciation in Wisconsin time. Glacial erosion has resulted in many of the bedrock exposures having rounded "whaleback" morphologies. In some areas glacial erosion has oversteepened faulted outcrops resulting in rock talus at the foot of slopes. Stress relief joints, resulting from exfoliation, are present in some areas due to unloading after glacial retreat. Differential weathering, glacial scour, and erosion by melt water runoff has resulted in a highly variable and undulating top of rock surface in many areas.

SLOPE STABILITY PROBLEMS

Slope stability problems have been the result of at least three geologic influences: structure, glaciation, and lithology.

<u>Structure</u> - Discontinuities in the rock mass along the project alignment induced by movement along the Ramapo Fault are the controlling factors governing slope stability. Exfoliation joints are properly placed in this category, but are discussed later since they play a minor role in overall slope stability. The primary structural discontinuities of concern have been joints, shears, cataclastic foliation and faults.

Probably the single most dominant structural feature observed along the alignment has been the cataclastic foliation related to the Ramapo Fault. The grain imparted to the rocks by the fault has been described by Ratcliffe (Reference 3) as having grooved surfaces with slickensides, rusty weathering, and chlorite coating. Ratcliffe has also noted that extensive fracturing is limited to within 100 to 200 meters of the fault. The rock grain described by Ratcliffe has been well documented and verified as a result of the excavation of the slopes for this project.

Where cuts have been within close proximity to and paralleled the strike of the fault, entire cut heights have been controlled by the dip of the cataclastic foliation and shears associated with the Ramapo Fault. This condition occurred in two of the largest cuts on the project, with heights ranging from 90 to over 200 feet. Joint surfaces in both these areas were noted as being continuous for 100 to 300 feet. Stabilization attempts of the entire slope heights against plane failures at either of these locations would have been a monumental task. In both cases the cuts were excavated to an angle closely parallelling the dip of the cataclastic foliation. Additional stabilization was needed only for localized instabilities in either cut.

Slope instability due to the presence of adversely dipping set(s) of discontinuities has required support to prevent potential plane and wedge failures. Wedge failures have been produced by the intersection of two or more joint sets as well as the intersection of joints and foliation. In all cases, potential wedge failures produced by joints and/or foliation planes have been confined to localized areas and have not controlled overall slope stability to any great extent. Wedges up to 350 cubic yards have been identified and stabilized with rockbolts and/or dowels.

Plane failures along joints or foliation have for the most part been small, local occurrences, with one exception. This failure involved sliding along a 40° to 45° dipping, smooth, planar joint set that is perpendicular to foliation and has the same strike as the alignment. This resulted in destabilization of the entire cut height (approximately 75 to 80 feet) for a length of about 300 feet. This orientation was not found in field mapping or exploratory drilling. The cut design was originally 1H:6V (80 degrees), but within a matter of weeks the downslope migration of the rock above these planes was evident. The cut was redesigned to 1H:1V (45 degrees), and no further stabilization was required.

Large scale slope stability problems as a result of faults or shears that required major stabilization or protection methods have only occurred in one area. This is an area with steeply dipping, weak, brittle faults that strike approximately 40 degrees to the cut orientation. This zone was treated with shotcrete and dowels for the entire cut height (up to 60 feet) to prevent destabilization of adjacent rock by weathering and erosion along the faults. This area also contains the only large scale wedge instability as the result of the intersection of two low angle faults, each having an average dip of approximately 39 degrees. The intersection of the faults plunges 35 degrees into the cut at an angle of 75 degrees to the roadway. Both surfaces contain weak, sheared, clayey gouge up to four feet thick. Total height of the cut in this area is approximately 100 feet, with the intersection of the wedge daylighting approximately 30 feet above the toe of the slope. The size of the wedge was estimated to be approximately 12,000 cubic yards. It was recommended that the wedge be removed due to the cost of stabilization and concern for long-term stability and safety. At the time of this paper, the layout and drilling for the excavation of this wedge is commencing.

One other area of note is located at the end of the alignment in Mahwah, New Jersey. The sidehill cut is approximately 100 feet high and was originally designed to be cut at 1H:6V (80 degrees). After blasting of the first lift of the cut, a persistent set of joints dipping approximately 40 degrees into the cut was noted. The joint surfaces are slickensided and in some instances are underlain by very weak, coarse grained, syenite. Plane failures occurred immediately upon excavation. As excavation proceeded and the cut was more thoroughly investigated, a polished discontinuity surface dipping 40 degrees into the excavation was exposed. This discontinuity surface has from two to twelve inches of what appears to be glacial till (diamict) infilling. The seam was found on either side of the outcrop and appeared to be continuous for the entire height of the cut. Continuity of the seam between holes up to 100 feet apart was observed during drilling.

Projecting the trace of this seam indicates that it "daylights" near the toe of the slope. This could destabilize the entire cut for its full height and involve nearly 85,000 cubic yards of rock. The origin of this seam is unknown; possible explanations for the mass that overlies the discontinuity are that it is: 1) a landslide block, 2) part of an allochthonous mass put in place by thrust faulting, or 3) deposition of a very large boulder by glaciation. Excavation of this slope is to resume in the fall of 1991. The slope will be excavated on an overall 40 degree angle to accommodate this structure.

<u>Glaciation</u> - Glaciation has occurred throughout the project area. Rebound, as a result of glacial retreat and unloading has formed exfoliation joints that parallel the ground surface. These exfoliation joints are not consistent in their occurrence in the project area. Exfoliation joints were observed to generally dip between zero and 30 degrees and are present to depths of up to 15 feet below the ground surface.

Whereas exfoliation joints have not been responsible for large scale slope instability, they have caused construction/excavation problems. Backbreak, breakage of rock behind the row of pre-split holes, along the crest of cuts with exfoliation joints tends to occur due to explosive gases venting along the weak joint planes. This results in unstable, blocky rock conditions along the top of cut which require the contractor to regain access to the crest of the slope. In areas of limited right-of-way, backbreak all but eliminated the planned top of rock bench. Soil slopes above the top of rock had to be re-graded as a result of backbreak.

Glacial scour, differential weathering, and melt water runoff has left a very irregular top of rock profile in many areas. This condition made establishing the designed top of rock cut line a challenge for the contractors' layout and drilling operations, but did not pose a threat to overall stability. In some instances, the top of cut line had to be moved because the undulating, glacially polished slopes made drilling access impossible. Because of the unpredictable top of rock profile in some areas and the difficulty in portraying these conditions on the design drawings, the contractor and engineering geology staff had to work in concert using air track probe borings on 20 or 25 foot centers to better delineate top of rock.

Lithology - During the initial design investigations for the project, drilling and field mapping identified areas where poor rock conditions in the upper part of slopes might exist. These were areas where RQD tended to be low, generally less than 60 percent. Two areas where this condition did exist are near the I-287 and Route 23 intersection in Riverdale, New Jersey. The rock types present are amphibolite gneiss and chlorite-talc schist in one cut, and pegmatite gneiss in the other.

In both cases, the original design slope was to be 1H:6V (80 degrees). Approximately the upper one-third of both cuts (maximum cut heights were 65 and 140 feet) had to be graded to a rubble slope (1.5H:1V or 33.5 degrees). The deep weathering in these areas precluded rockbolting due to the associated weak rock mass. Rock in both of these areas is thoroughly weathered and includes iron stained fracture surfaces in the upper 40 to 60 feet. The lower reaches of these cuts were held steep as the rock quality improved significantly with depth.

SLOPE STABILIZATION SPECIFICATIONS

After Golder Associates Inc. was brought onto the construction phase of the project, we were asked to review and update specifications related to rock stabilization to make sure the most up-to-date methods were being used. Contract items pertaining to slope stabilization and protection included: Rockbolts, rock dowels, high capacity anchors, shotcrete, horizontal drains, and wire mesh. The most important changes made were to the shotcrete and rockbolt specifications, the two most widely used items for stabilization and protection.

Shotcrete - The original shotcrete specification called for welded wire fabric to be installed prior to shotcrete application. This would have required two separate operations: 1) drill and install anchor bolts, then install wire fabric, and 2) application of shotcrete. Installing the welded wire fabric would have posed a challenge in trying to make the mesh conform to very irregular cut faces that include sharp projections out of the plane of the slope in localized areas. Large amounts of additional shotcrete would have been needed to fill in deep pockets behind the wire fabric in order to get the minimum cover over the front of the fabric.

As an alternative, it was proposed to utilize steel fiber reinforced shotcrete. This method does not require the contractor to install dowels and wire fabric as excavation proceeds and the minimum thickness of the application does not require large voids to be filled. Steel fiber reinforced shotcrete allows for excavation to proceed, if desired, and then return to apply shotcrete over the entire slope length. The one exception to this is the drilling of weep holes, which is preferably done as excavation proceeds.

It was left to the contractors' option to use either the dry or wet mix method and both methods have been used. Shotcrete applications have been done full slope height as well as by excavation lifts. All contractors on the project have sub-contracted shotcreting to specialty firms.

Rockbolting - Specifications for rockbolts (tensioned) originally called for a two-stage grouting procedure to anchor the bolt. The steel reinforcing bar was to be installed along with cement grout over the anchorage length of the bolt. The grout was then to cure for 14 days which also meant that blasting within 100 feet of the rockbolt could not be done during the curing period.

This method was obviously not compatible with a project of this magnitude where up to 8000 cubic yards of rock per day has been excavated.

A new specification was submitted which called for the option of using either mechanically anchored or polyester resin anchored bolts. The reason for specifying both types is due to the problem of trying to spin a bolt more than 20 to 30 feet through resin cartridges while ensuring proper mixing of the two-part compound. It was left to the engineer's choice as to what type of bolt to specify for installation. If rockbolt lengths longer than about 30 feet were required the mechanically anchored bolt would have been more desirable. So far, all of the rockbolts installed on the project have been the polyester resin type.

A clause in the specifications noted that access for rockbolt installation and inspection more than 12 feet above the working bench at the time of marking would be paid for by NJDOT. The contractor was responsible for providing access to rockbolt locations below the 12 foot limit.

SLOPE EXCAVATION SPECIFICATIONS

In the desire to minimize the cost to NJDOT for installing stabilization/protection items on the slopes, drilling and excavation specifications were modified and some new items were added. This was also done to allow the engineering geology staff to inspect the slope conditions as excavation proceeded and make recommendations for support in a timely fashion.

Drilling and Excavation - In order to keep up with the volume of rock being excavated and the cut faces being exposed, drilling lifts were limited to 35 feet. Exceptions to this were made with the engineer's approval when the first lift of rock along the crest of slopes was irregular in order to allow the contractor to maintain a consistent bench elevation through an area. Excavation lifts along the final cut face were limited to 15 feet to allow for inspection of the geologic conditions by the engineer. If conditions were such that remedial work was needed, excavation was to stop when the engineer notified the contractor.

Exploratory Drilling - Due to the very irregular top of rock profile in many areas (as previously discussed), a bid item for an airtrack drill was added to the specifications. This provided the engineer a means of investigating depth to top of rock along the proposed top of cut line. Information from this drilling was used to revise the top of cut layout and determine whether retaining structures were needed for soil slopes above the rock cuts. Included in this item were a compressor and two drill operators.

Scaling - Due to the number of geologic and structural variations along any given cut, scaling of weak rock and soil along the crest of a cut or removal of loose blocks or wedges in the cut face was needed on a daily basis. Before construction line items to perform this type of work were implemented, scaling was done under a force account, which proved costly. With the line items available, the engineer could select and direct appropriate activities at a reasonable cost. The backhoe item was used to excavate test pits to investigate top of rock depth or fault conditions. Other items used for stabilization access were bulldozers and front-end loaders to build rock-fill ramps. Two contractors used hoe rams mounted on a backhoe which proved to be quite successful in removing potential plane and wedge failures.

Scaling/Stabilization items were specified in the following manner:

Item Description	Unit
Bulldozer D-8	*Man Hour*
Backhoe 235	*Man Hour*
Front-End Loader 988	*Man Hour*
Airtrack Drill and Compressor	*Man Hour*

APPROACH TO STABILIZATION

The methods employed to produce stable rock slopes have generally followed the pattern of data collection, analysis, development of stabilization options, and implementation of the preferred option. In many cases involving larger scale stabilization requirements, the approach and implementation of remedial measures has involved a joint effort with input from the NJDOT resident engineers, engineering geology staff (NJDOT and Golder Associates Inc.), and the contractor.

Data Collection - As final cut faces are exposed, the engineering geology staff are present to observe geologic/structural conditions. Once an area is noted as having potential stability problems, data collection on discontinuity shape, roughness, persistence, aperture, infilling, water conditions, and spacing was undertaken. In some instances mapping was done on mylar overlays of photo mosaics taken of the cut area.

Analysis - Failure modes on the cuts have been limited to planar and wedge types; toppling and circular failures have not been encountered. Analyses of planar failures have been carried out under static equilibrium on a "sliding block". Wedge failures have been analyzed with computer programs with which a number of iterations can be quickly run on a range of parameters. The wedge program has also been very helpful in determining the volume of a given wedge, which is usually difficult to estimate with accuracy.

Among the most important parameters in a stability analysis are the roughness and water pressures on the discontinuity surfaces. The friction angle of joints as determined by laboratory testing is approximately 36°. Roughness on a site-specific basis was either measured or estimated. The roughness angle generally ranges from 4° to 8°, resulting in a total (effective) friction angle of 40° to 44°. Cohesion was assumed to be zero for all stability calculations because site-specific test results were never developed. This assumption errs toward a more conservative design. Where water conditions were noted with respect to an unstable block, drain holes were installed to prevent build-up of hydrostatic forces. Movement of water in the rock occurs along discontinuities (secondary permeability), which are mainly joints and shears.

Development of Options - The available methods for slope protection included shotcrete and wire mesh. Slope stabilization items available included rockbolts, rock dowels, high capacity anchors, scaling, and re-grading.

Slope protection was required in areas where weak rock conditions are present due to faults and/or shears or in highly fractured/jointed rock. Shotcrete was used in most of the large, brittle fault zones. In some instances this involved cut heights up to 80 feet. In some areas, substantial weathering of fault zones near the original ground surface resulted in extremely weak rock slopes. Rock dowels on a six foot by six foot pattern, eight feet long with a six inch stick-up were installed in these weak zones to provide a structural framework for the shotcrete.

Wire mesh has only been used on a few occasions thus far. In areas of highly jointed/fractured rock, wire mesh has been installed. The option to either drape the mesh over the slope or to provide an active restraint by pinning the mesh to the cut face is left to the engineer's discretion in the specifications.
Slope stabilization options were developed for large scale instability conditions. The preferred options were either slope reinforcement or re-grading. Reinforcement involved the choice between installing rockbolts or high capacity anchors. Wire mesh and/or shotcrete was used in conjunction with the rockbolts if the rock mass has weak zones. The option of re-grading the slope to a flatter angle or constructing benches was the alternative to reinforcement.

Quantity and cost estimates for all the various options were calculated. Other factors considered were the concerns of the resident engineer and the contractor. These included: long-term stability, maintenance, right-of-way restrictions, and construction sequencing/disruption. The option to re-grade slopes has generally been preferred by NJDOT. The areas where re-grading has been done (some previously discussed) have usually been where entire cut heights of up to 200 feet were potentially unstable. Reinforcement for these locations would have been an enormous undertaking. Re-grading of slopes has been based on accommodating the natural angle of discontinuities or fabric in the rock to the cut angle of the slope. The cost of re-grading versus reinforcement has varied between locations, but the long-term stability and minimal future maintenance of the slopes has favored re-grading.

Supplemental Slope Design Methods - There are two other techniques that have been used in analysis of rock slope stability on this project that are worth noting.

Golder Associates Inc. was asked to provide an assessment of requirements for rockfall catchment areas along the project alignment. A computer program to simulate rockfalls was written to predict rockfall patterns. It was found that a determining factor in the behavior of rockfall is the roughness of the slope face. Site-specific measurements were taken early-on in the project to provide a basis for initial catchment requirements. Rock catchment fences and gabion walls will be constructed as protection devices to prevent rocks from reaching the roadway. Final recommendations for dimensions of catchment areas will be made on completed slopes in the near future.

A laser profiling system was used to define the limits of a large wedge (approximately 12,000 cubic yards) previously noted. The profiling system was able to provide a series of cross-sections through the wedge which helped to better define its volume as well a provide the blaster with a better estimate of burdens for the blast design. This system is not inexpensive, but could provide valuable information for analysis of existing rock cuts where access to the face is difficult.

SUMMARY

Construction of nearly nine miles of rock slopes for the extension of Interstate 287 in northern New Jersey is soon to be completed. A project of this size has required on-site engineering geology staff to evaluate and design rock slope stabilization and protection techniques as construction proceeds.

This area is within the New Jersey Highlands, and rock types include a variety of Precambrian Age Gneisses with varying mineral assemblages. Foliation of the rocks strikes northeast and dips steeply to the southeast. This generally follows the trend of the Ramapo Fault, the dominant structural component of the region. Slope stability has been controlled by structure, lithology, and glaciation, with both plane and wedge type failures having been identified.

Movement along the Ramapo Border Fault has produced a grain in the New Jersey Highlands rock remarkable for its persistence. North of Route 23, the resulting discontinuities are generally shear joints of 100 to 300 feet in length while south of Route 23, cataclastic foliation in the form of phyllonites and mylonites is the primary control on slope stability. The cuts that have been most controlled by the Ramapo grain have been composed of rocks identified as metasediments or cataclastics. Meta-igneous rocks do not exhibit the same orientation of persistent discontinuities although sub-vertical tension fractures of approximately 100 feet are present. Where the trend of a cut has parallelled the strike, and been in close proximity to the Ramapo Fault, entire cut heights (up to 200 feet) have contained the Ramapo grain. The dip of the grain has ranged from 40° to 60° to the southeast. Borings taken in Riverdale, New Jersey have intersected the Ramapo Fault and the dip was measured at 56° southeast. This is the same angle found in the grain of most of the rock from Montville to Mahwah, New Jersey. The exception to this is where cuts are on a dogleg away from the Ramapo Fault and cut angles as steep as 65° to 80° have been possible.
Slope design has accommodated the natural structure of the rock to the extent possible to provide long term stability and minimal maintenance. Attempts at benching in some areas have not proven successful.

Where slopes had to be kept steeper than the inclination of discontinuities due to limited right-of-way, a variety of stabilization and protection methods have been implemented. All slope stabilization and excavation specifications were revised and rewritten in an attempt to minimize construction delays and permit the most up-to-date techniques to be utilized.

ACKNOWLEDGEMENTS

The author wishes to thank William A. Lounsberry of the New Jersey Department of Transportation for his critical review and comments of an early draft of this paper. I would also like to thank Kenneth H. Rippere of Golder Associates Inc. for his critical review and assistance in the preparation of this paper.

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ROCK SLOPE INVESTIGATIONS AT SELECTED HUDSON VALLEY SITES

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ABSTRACT

In response to deterioration of rock slopes along their right-of way, the New York State Thruway Authority has begun an aggressive remediation program that includes a prioritized plan for investigations and repairs. This paper presents a summary of the findings from an investigation of conditions at four selected slopes located over a 75-mile stretch south of Albany. Rock varied from heavily fractured metamorphics in the south to gently tilted, but well weathered, sedimentary rock in the north. The investigations were hampered by the steepness of the slopes and the steady flow of traffic. Options for remedial work at some sites were limited due to very tight right-of-way restrictions. The condition of these specific slopes and the potential impact of even small rocks on high speed vehicles lead to recommendations for the use of aggressive remedial measures.

INTRODUCTION

The New York State Thruway cuts through many hills and exposes a wide range of geologic conditions. More than three decades of weathering has affected all of the excavated rock slopes producing a variety of slope stability problems requiring different solutions by the New York State Thruway Authority (NYSTA). Several sites are described below.

The rock cuts were created approximately 35 years ago during construction of the New York State Thruway (Interstate 87). They were excavated using contemporary mid-1950's blasting techniques; smooth blasting techniques, such as pre-splitting, were not in common use in this area at that time. The similar age of the rock cuts provides a unique opportunity for assessment of long-term rock face stability.

The four case studies presented have been selected because they represent a range of varied rock slope remedial strategies in diverse geologic settings. They comprise four of the rock slopes evaluated during a rock slope inventory conducted for the NYSTA in 1988 (Burke and LeFevre, this symposium), investigated further in 1989-90 (Dunn Geoscience Corporation; 1989, 1990a, 1990b, 1990c) and remediated by the NYSTA in 1990-91. The higher priority slopes identified during the inventory in 1988 were remediated previously in 1988-89, typically by a combination of scaling, wire mesh application, and catchment ditch enlargement.

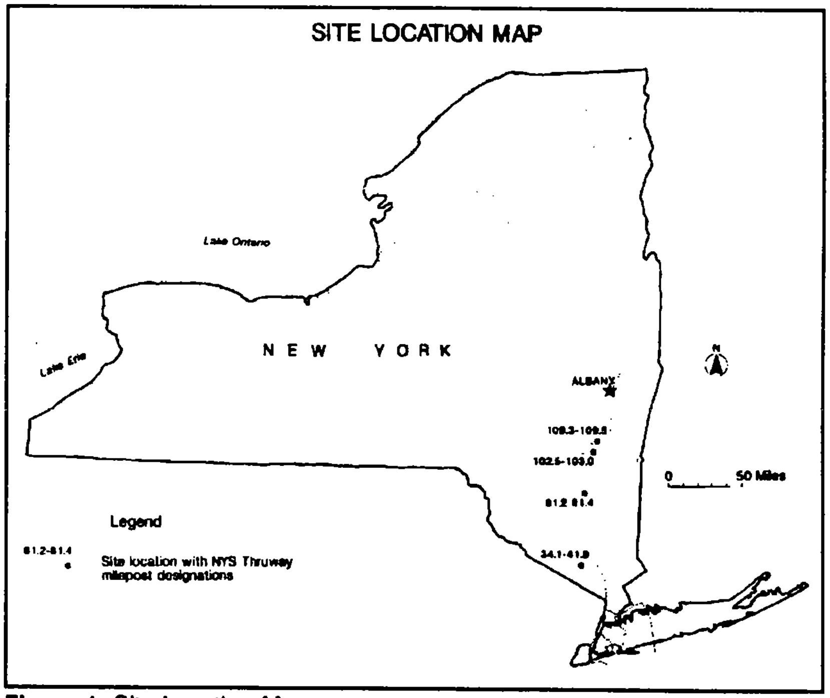

Figure 1: Site Location Map.

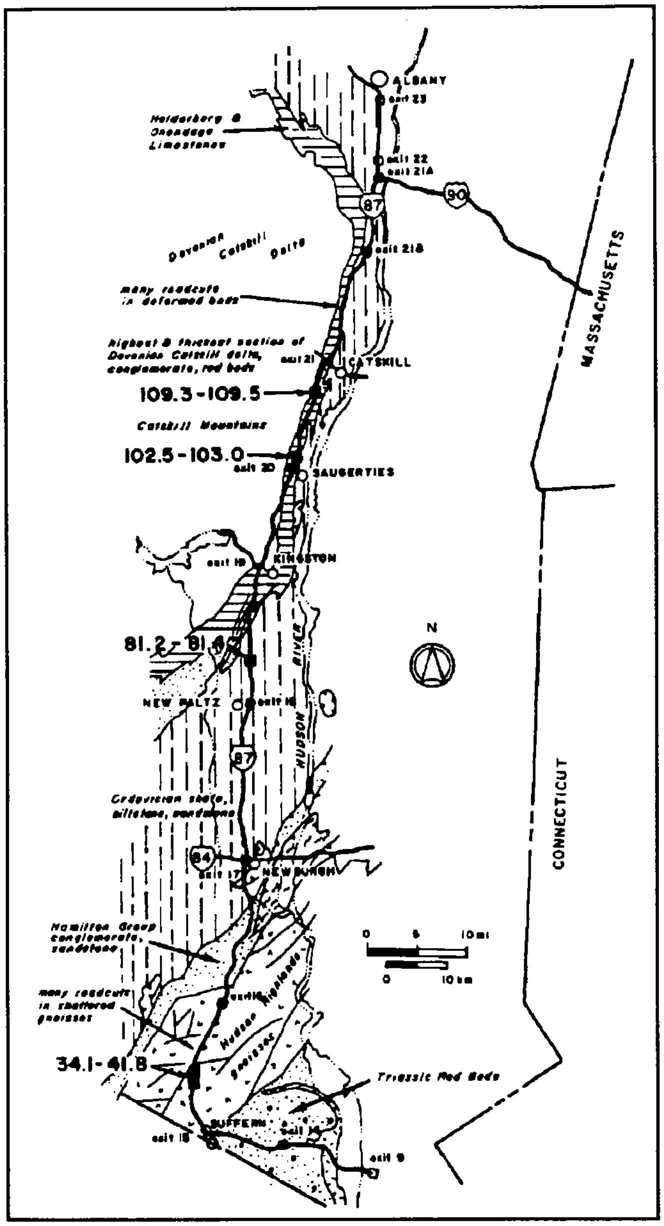

Figure 2: Geologic map (adapted from Van Diver)

The rock cuts are located within a 75-mile long south-north stretch of the Thruway (Figure 1) extending between Suffern (Thruway exit 15) and Catskill (Thruway exit 21). This section of the Thruway extends geologically from the metamorphics and igneous intrusives of the Hudson Highlands, through Ordovician sedimentary clastics of the Hudson Valley Lowlands and Silurian-Lower Devonian carbonate and siltstone strata of the Helderberg and Tri-States Groups, and into Middle Devonian clastics of the Catskills (Figure 2). The four sites discussed herein are identified by Thruway milepost markers as follows:

34.1 - 41.8 northbound (east) side; Hudson Highlands gneisses.

81.2 - 81.4 northbound (east) side; Hudson Valley Lowlands siltstone and shale.

102.5 - 103.0 southbound (west) side; Helderberg Becraft through Port Ewen Formations; limestones with interbedded shales.

109.3 - 109.5 northbound (east) side; Tri-State Schoharie Formation; siltstone.

The main concern at these sites is the incidence of rock falls which pose a potential safety hazard to passing motorists and increase Thruway maintenance. Close examination during site investigations also indicated a potential at some of the slopes for relatively large toppling, wedge and sliding block failures to occur over the long term. The process for evaluating alternative remedies considered many factors, the relative importance of which varied at the four sites. Particularly challenging major factors included property right-of-way restrictions, maintenance of traffic safety and flow, aesthetics, and highway planning and maintenance requirements.

Following is a brief description of each cut prior to remediation, the stability problems it presented, and the selection process for the recommended remedial approach. The sites are presented in order of increasing complexity.

SITE 102.5 -103.0 - HELDERBERG GROUP: BECRAFT THROUGH PORT EWEN FORMATIONS; LIMESTONES WITH INTERBEDDED SHALE

Site 102.5 - 103.0 is located approximately one mile north of Thruway exit 20, Saugerties. The northern end of the cut is contiguous with an exit/acceleration lane for a southbound rest stop/parking area. Of the four rock cuts discussed, it is the only one on the southbound side of the Thruway. The corresponding larger rock cut adjacent to the northbound lanes also has been remediated recently, and is not addressed in this paper.

The approximately 2600-foot long cut is located at the crest of a hill and curves gently from a bearing of North 0° East at the north to North 27° East at the south end. The height of the near-vertical cut ranges from about 10 feet at the ends to a maximum of 35 feet along the middle sections. The roadway grade rises steadily towards the south roughly parallel to the strike so that the rock units exposed at roadway grade at the south end of the cut on the opposite side of the hill stratigraphically underlie those rock units to the north.

The original catchment ditch between the rock cut and roadway varied uniformly in width from about 8 to 12 feet. The Thruway side of the ditch had a guard rail and was steep, approximately 1 V:1 H. Most talus observed in the ditch was accumulating in discrete areas in the high central part of the cut. The talus ranged from relatively fine grained weathered shale to angular blocks of limestone as large as a cubic yard in volume. According to the Thruway Authority, the ditch was last cleaned out in 1988; therefore the debris had accumulated within approximately a one-year period.

The Thruway right-of-way line extends back approximately 40 to 60 feet behind the face of the rock cut. The land surface above and behind the cut is a flat to very gently sloping natural surface covered with trees typically less than 8-inches in diameter, but ranging up to 18 inches. At the southern half of the cut, the overburden soil is thin with frequent bedrock outcrops. This area contains numerous boulders and evidence of a few open solution-enlarged joints ranging from several inches to two feet in width.

The bedrock exposed in the stratigraphically lower southern half of the rock cut is the Becraft Formation, a relatively massive, coarsely crystalline, highly fossiliferous white-weathering limestone susceptible to solution activity. The overlying exposed rock units progressively to the north, are the relatively thin Alsen and thick Port Ewen Formations, interbedded limestones and dark gray calcareous shales. The contacts between the formations are relatively distinct. The shaliness of the Port Ewen provides most of the stability concerns at the road cut. The shale interbeds are typically two to six inches thick, but range up to two feet in thickness. This shale is highly susceptible to weathering and in many areas differential weathering has resulted in erosion of the

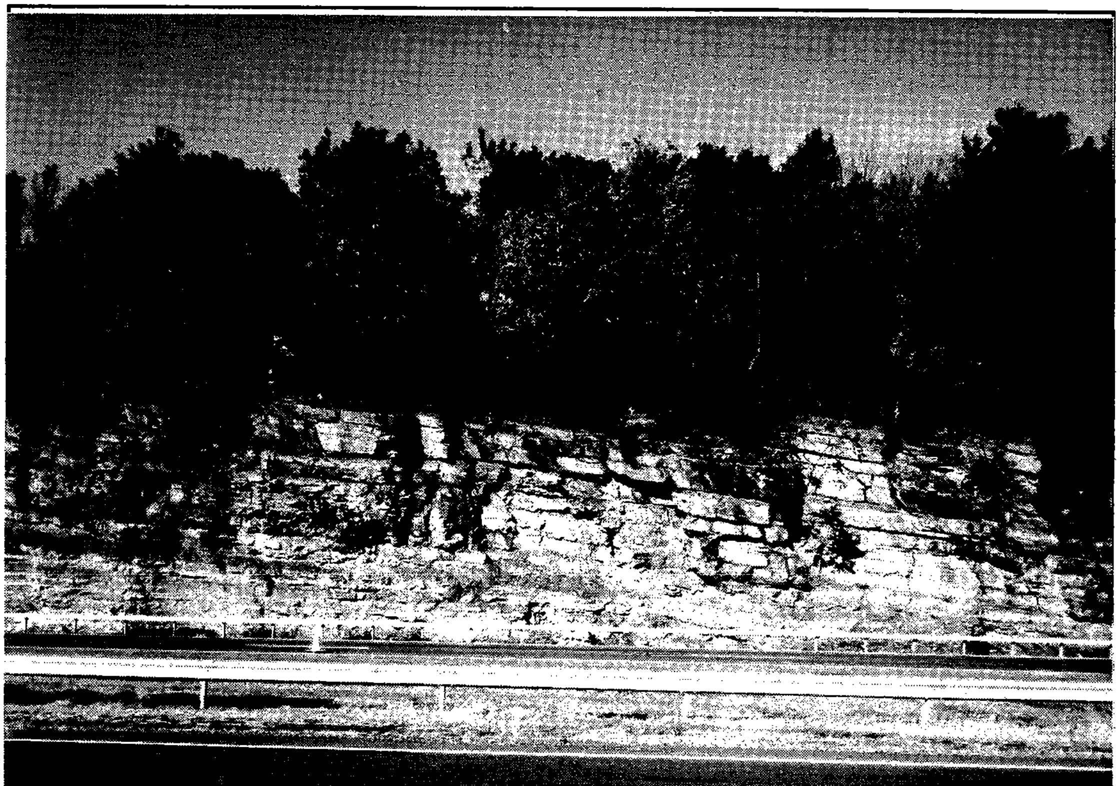

Figure 3: Photograph at milepost 102.5-103.0 showing undermined blocks of Port Ewen limestone; view looking west.

weak shale and undermining of the overlying more resistant limestone blocks. Where undermining is well-developed, especially along the crest of the cut, limestone blocks isolated by intersecting joints and bedding planes have rotated towards a potential toppling condition (Figure 3).

The principal structural discontinuities observed were in the interbedded limestones and shales. These discontinuities consisted of the bedding planes and a set of near-orthogonal joints. The beds have an average strike of about North 15° East and dip 15° to 31° to the southeast, generally toward the roadway. The steeper dips are at the northern end of the cut. The joints trend nearly parallel and normal to the cut face and are steeply dipping. These joints intersect the bedding planes and divide the limestone beds into blocks and wedges. These blocks range in volume from less than one cubic foot to greater than 10 cubic yards. At several locations, especially in the upper part of the cut, blocks were observed to be detached along the steeply dipping strike-parallel joints, and rotated towards the Thruway.
Detached blocks pose a potential threat of toppling, translational slides and wedge failures. Most of the falling debris, including boulders eroded from the overburden and blocks fragmenting upon impact, was retained in the catchment area. However, any debris reaching the roadway would pose a special hazard because visibility of the roadway is limited and attention could be distracted as cars exit the rest area and accelerate along the rising curve adjacent to the cut.

Remedial Alternatives at Milepost 102.5 - 103.0

Consideration was given to the following remedial alternatives; (1) rock bolting, (2) wire mesh containment, (3) shotcreting, (4) scaling, and (5) large scale rock removal by blasting. Because the disintegrating beds of shale had undermined numerous large blocks of limestone, (especially on the crest of the cut), many of these common remedial measures were considered inappropriate.

Rock bolting would have been complicated by the large open joints, the persistent fracturing of the blocks and the susceptibility of the shale beds for further disintegration. Where the drilling for rock bolt installation encountered large, open joints, various types of sleeves could have been inserted to span the void and facilitate grouting. The fracturing and jointing of the blocks could have been dealt with by using extra bolts and strapping or heavy mesh with shotcrete in problem areas. However, continued deterioration of the weak beds would have led to settlement of the large blocks and the imposition of shearing stresses across the rock bolts.

Fastening wire mesh or cyclone fencing over the face of the cut would have helped control smaller blocks but would not have been effective containment for the large uppermost blocks which were believed to be the main threat. More detached blocks are likely to develop because of the susceptibility of the limestone to solutioning and of the shales to slaking.

Shotcreting has been used successfully as a preventative measure on slopes with similar conditions. At this site, it would first have been necessary to clean the surfaces to be shotcreted and then to shoot several layers onto the deteriorating seams to form a wedge to support the large blocks above. In addition to serving as a structural element, shotcrete helps protect rock from some forms of weathering. However, since many of the larger detached blocks already showed significant movement, this approach might have been too late for some blocks. Slaking, solutioning and frost action were likely to continue the undetected development of openings behind the shotcrete. Eventually, the shotcrete could have been left as the primary support for the front of a block and would be susceptible to crushing. With time, weathering processes would attack the shotcrete as well. Non-technical factors must also be considered. Many motorists appreciate the sight of the rock slopes on a long trip and reportedly have objected strongly to having the scenery "blemished" by shotcrete.

Scaling is defined here as the removal of loose rock from slopes. Where rock is loose and unstable, individual rocks can be manually removed with the aid of a scaling bar. A scaling bar is typically around four feet long and is bent a few inches from one end which is flattened to a chisel point. The flattened end can be inserted into open joints to pry loose rock from the face. Long-reach backhoes or Gradalls may be used for scaling in areas where large quantities of loose rock are involved. Large loose blocks may be removed with a backhoe bucket, a Gradall hook, by pulling with a cable attached to a vehicle on the ground or by dental blasting. Blasting used to remove individual large blocks generally involves placing a charge in an opening behind the rock and packing it in place with sand or sand bags. Occasionally, drilling may be required to introduce the charge. The major advantage of scaling at this site is that it would have removed the rock which is in imminent danger of falling.

However, there are several disadvantages to scaling. Scaling is a temporary remedial measure. With time, stable rock will be attacked by weathering, solutioning, root action and other processes, which produce a new generation of unstable rocks. Some areas of this cut were higher than 25 feet which is greater than the reach of most backhoes or Gradalls. This would have required that heavy equipment work from above the cut, thereby disturbing the overlying vegetation and soil cover. Scaling, if used as the sole remedial measure, would not have provided additional catchment for rockfalls.

In a situation where it is not practical to prevent rock from falling from slopes, a catchment area is sometimes a reasonable solution. A catchment area is defined as a zone at the base of the slope where rock may safely fall without endangering the public. The catchment area may include a wide ditch with a depth sufficient to prevent rocks from rolling out after they hit. According to guidance provided by the United States Department of Transportation's Federal Highway Administration in the 1981 publication "Rock Slopes," the older ditch configuration at this site was inadequate over most of the rock cut length. The most direct way to provide a catchment area is to blast the slopes back.

One of the major disadvantages of rock blasting is the possible risk to or interruption of Thruway traffic. During blasts, traffic must be stopped a safe distance away from the work area. A detour is usually needed in order to provide a safe corridor for the rock removal. Delays to traffic are likely to occur while thrown rock is removed from the Thruway. The blasting must be properly performed to minimize damage to the remaining rock mass.

Recommended Alternative at Milepost 102.1 - 103.0

Large scale rock removal at this site was the recommended remedial method because the following conditions existed.

1) There was a desire to increase the lateral distance from the travelled Thruway lanes to the slope.

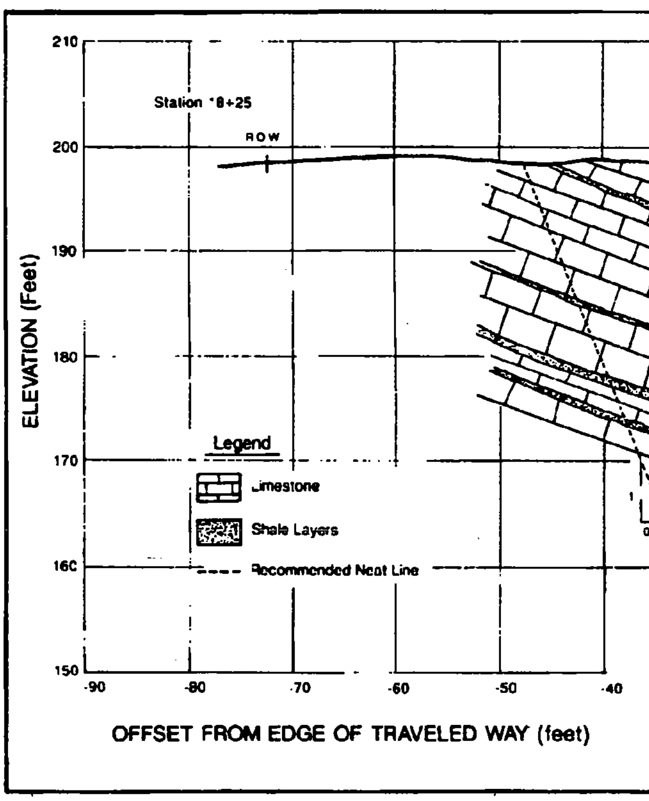

Figure 4

TYPICAL SECTION
MILEPOST 102.5-103.0

NEW YORK STATE THRUWAY AUTHORITY
ULSTER COUNTY, NEW YORK

March 22, 1990

Figure 5: Photograph at milepost 109.3-109.5 showing large detached block at base of Schoharie siltstone cut and discontinuities dipping toward roadway; view looking east.

2) There were numerous potential stability problems, each encompassing tens of cubic yards of material.

3) Sufficient right-of-way existed for modifying the slopes to a safer geometry.

4) Air track access to the overlying slope was good to excellent.

The recommended geometry for rock removal by blasting included a catchment area at the toe of the slope which was a minimum of 30 feet wide (Figure 4). This space was intended to catch falling rock and prevent it from reaching the Thruway. The recommended final slope had a minimum slope angle of approximately 68° from horizontal in areas where the maximum slope height of the rock face would be approximately 35 feet. For this combination, the Federal Highway Administration's recommended catchment area would consist of a ditch approximately 16 feet wide and 5 feet deep. The deep ditch would be intended to prevent boulders from rolling out into traffic.

The presence of a five-foot deep ditch adjacent to the Thruway would be a definite hazard in itself, and should not be considered without a guardrail. Because of the guardrail's cost and maintenance requirements, the NYSTA expressed a desire to use wide, obstacle-free shoulder areas instead. This would require use of a catchment area without an abrupt ditch. To accommodate this, the recommended catchment width was increased from 16 feet to 30 feet.

To minimize block loosening and damage to final rock faces, it was imperative that appropriate smooth blasting techniques be used. These techniques consist of either 1) pre-splitting, where the holes along the finished face are closely spaced, lightly loaded and fired prior to the production shot, or 2) smooth blasting, where the holes along the finished face are lightly loaded, closely spaced and fired after the production holes. In our experience, smooth blasting has generally produced more competent finished faces than presplitting has. The blast gases generated by a pre-split operation are in a confined rock mass and must expand outward through existing cracks. Frequently, this shears off the top edge of the finished face or loosens or fractures rock in the finished face. We therefore recommended that later-fired smooth blasting be used rather than pre-splitting. While both procedures tend to shear the rock in a neat plane, the smooth blast sequencing provides a free face towards which blast gases can expand.

SITE 109.3 - 109.5 - TRI-STATES GROUP: SCHOHARIE FORMATION; SILTSTONE

Site 109.3 - 109.5 is located along the northbound lanes eight miles north of Thruway exit 20. The northern end of the cut is contiguous with a Thruway maintenance area, reportedly an abandoned proposed exit ramp, now characterized by the presence of large stockpiles of crushed stone gravel. The Thruway curves west at this point, away from the rock cut.

The approximately 1100 foot-long rock cut is oriented North 20° East. It had an irregular face that ranges in inclination from near-vertical to 30 degrees, with occasional overhangs. The height of the cut approached a maximum of about 60 feet in the central part of the site, and the sloping (1 vertical on 3 horizontal) hillside above the cut continued to rise for an additional 40 feet. A few piles of talus have accumulated on the less inclined portions of the slope.

The original catchment ditch varied in width from 6 feet (south end) to 30 feet (north end) and contained sparsely distributed gravel-size talus. Occasional boulders up to one foot in diameter were noted near the toe of the shalier northern end of the cut. A distinctive landmark at the northern end was an approximately 30 cubic yard boulder situated near the base of the cut (Figure 5). This ominous-looking block reportedly became dislodged and fell approximately 12 years ago. It was left in place because of its large size and relatively safe distance from the roadway.

The right-of-way line is located on the hillside between the rock cut and hill crest above. The line is parallel to and approximately 100 feet behind the toe of the slope. The hillside is vegetated by trees up to 10 inches in diameter. Overburden soil appears to be till containing boulders up to two feet in diameter. At several locations the soil was observed to be creeping and eroding over the top edge of the cut.

The bedrock was mapped as the Schoharie Formation, predominantly siltstone with a number of thin, dark gray shale beds at the north end. The siltstone weathers to a light to medium brown banded appearance. The contact between the Schoharie and the underlying Esopus Formation, a dark gray siltstone and shale with prominent slaty cleavage, is evident at the north end of the cut.

The rock cut exposes the west limb of a southward plunging anticline with axis striking approximately North 28° East. A secondary, less prominent anticlinal axis that trends approximately North 22° West cuts across the primary fold axis in the southern section of the cut. The beds dip steeply toward the roadway at the northern end of the cut; dips are more gentle at the southern end. Structural relationships suggest that beds may dip more steeply behind the face.

The predominant discontinuities are the bedding planes which have been enhanced by differential weathering and possibly by bedding plane slip, especially at the north end. Joint attitudes and spacing are variable along the cut and seem to be related to lithology and fold axes. Both strike-parallel and cross-cutting joints are present. The bedding, joints and cleavage planes have isolated several large blocks and resulted in a few overhangs near the crest that could potentially topple or slide down the slope. Approximately ten of these large blocks, similar in size to the fallen block described above, were reportedly bolted into the rock face during a program of scaling and rock bolting conducted about a dozen years ago. Recent observations suggest that these blocks are stable, but other partially detached and unbolted blocks ranging up to 18 cubic yards in size have also been identified.

The rock mass appears to be relatively stable. The principal stability concerns at this site are the erosion of the till overburden that could release embedded boulders and the undermining of blocks along weathered and weakened bedding planes. Both problems could result in rocks rolling or bouncing out onto the roadway.

Remedial Alternatives at Milepost 109.3 - 109.5

Consideration was given to the following remedial alternatives: (1) scaling, (2) systematic rock bolting, wire mesh and shotcreting, (3) ditch modification with suspended wire mesh for rock bounce prevention, (4) rock bolting, and (5) rock blasting.

There were several disadvantages to scaling the rock cut at this site. The rock face was scaled only a decade ago. Scaling was clearly a temporary remedial measure. Most areas of this cut were higher than 30 feet which is greater than the reach of most backhoes or Gradalls. The joint pattern at the north part of the cut is oriented such that if large blocks are removed, then the blocks lying directly behind those may also develop similar detachment joints and could have become dangerous, especially if left unbolted. The apparent stability of the blocks already bolted demonstrated that bolting could be successfully used.

Soil creep on the overlying natural slope will continue to convey soil and rock over the rock face. Scaling, if used as the sole remedial measure, would not have provided adequate catchment for rockfalls, particularly in the southern half of the cut. Therefore, a catchment structure and guardrail would probably have been needed to supplement the scaling operation.

Some slopes have been stabilized by systematic rock bolting, wire mesh application and shotcreting, which amounts to encasement in reinforced concrete. As a first step in this process, rock bolts are drilled into the face on a systematic pattern. Wire mesh is then fastened to the bolts as a covering over the entire face. As a final step, several coats of shotcrete are applied to thoroughly cover all exposed mesh. The shotcrete, a pneumatically applied fine aggregate concrete, serves to wedge blocks together and limit weathering of the rock and rusting of the metal. The mesh provides tensile reinforcement for the shotcrete. The rock bolts help hold the other components in place and tie the rock mass together to reduce the possibility of a massive slope stability failure. The system described above is used most appropriately on rocks that have above-average susceptibility to weathering and on carefully blasted slopes where the mesh may be conveniently fastened close to a uniform face.

The rock face at milepost 109.3 to 109.5 was very irregular. Any attempt to fasten wire mesh to the slope would have had regular gaps of several feet between the rock and the mesh. Shotcrete is usually applied in layers only a few inches thick. Numerous applications would thus have been required to develop the covering, and the shotcreting would have taken a long time to complete. With the difficulties and large quantities involved, the cost of this option would have been very high.

On rock slopes where small fragments ravel from the face, a remedial measure which has been used successfully on some rock slopes is to suspend wire mesh over the rock cut. The mesh is anchored at the top of the cut, drapes over the face and hangs down to nearly ditch level. The function of the wire mesh is to reduce the possibility that falling rocks will strike and bounce away from the slope toward the highway. Suspended wire mesh is most effective on slopes where particles are expected to emanate only from the rock face beneath the mesh. There must be stable anchorage for the wire mesh along the top of the cut face. The conditions at milepost 109.3 to 109.5 did not satisfy the above conditions. For example, the large block at the toe of the slope was testimony that the uppermost blocks were potentially unstable. Nearly vertical and slope parallel joints were present in many locations along the crest of the cut, presenting a likelihood of unstable blocks. Only a small amount of the fallen rock appears to have emanated from the rock face itself. Much of the potentially unstable rock was above the crest of the cut in the overlying soil.

Because of the large blocks that were apparent at this site, consideration was given to the use of rock bolts. Rock bolts are steel rods which are cemented or mechanically fastened into predrilled holes in the rock face. The protruding end of the rock bolt is usually threaded, permitting attachment of a steel plate and fastening nut. The function of rock bolts is to hold large blocks or masses of rock in place on a rock face at sites where it is impractical to remove the rock. Normally, rock bolting is most effective if the blocks which are to be bolted are greater than at least a cubic yard in volume, are relatively free from cracks and do not have a tendency to weather rapidly. If the stability of the entire rock face is suspect, bolts may be spread in a pattern over the entire face. Pattern bolting may not be necessary if discrete problem spots can be identified. Spot bolting is used where only a few bolts are needed in a specific block or localized zone on the cut face.

At milepost 109.3 to 109.5, spot rock bolting of large blocks was considered appropriate in conjunction with any of the previously discussed remedial measures. This was because of a potentially unstable joint pattern which was observed at several locations on the rock cut. Numerous large blocks (from 2 to 25 cubic yards) were resting on a sheared bedding plane and fold-slip joints dipping towards the Thruway. The overlying blocks were also in a position to topple or slide down the bedding plane towards the Thruway. If not removed, these large blocks would have to be bolted through their basal joints. The rock behind these facial blocks also had the potential to slide over the bedding planes so that bolting the facial blocks to those behind would have done little to reduce the likelihood of a bedding plane failure.

Recommended Alternative at Milepost 109.3 - 109.5

As stated previously, the major disadvantages of rock blasting are possible risk to or interruption of Thruway traffic and the detour needed in order to provide a safe corridor for the rock removal. However, large scale rock removal by blasting was recommended for the following reasons.

1) There was a desire to increase the lateral clearance from the roadway to the slope.

2) A potential deep-seated stability problem existed which was too large to manage using other remedial measures. The problem area could either be completely removed or removed to a sufficient distance to present a much lower risk to traffic.

3) Sufficient right-of-way existed or was available at reasonable cost.

In order to gain both lateral clearance for traffic and reduce the risk that large rock slope failures could reach the Thruway, the recommended modified cut geometry incorporated a sufficient catchment space and other supplemental measures such as rock bolting. The slope height and joint patterns presented potential deep-seated problems, especially if the rock slope was modified even slightly from its present configuration. The lowermost blocks keyed the overlying blocks into place. The lowermost key blocks dipped beneath the existing toe of the slope and would be removed by rock excavation. Bolting was therefore recommended to support the new geometry at this location. The most important safety aspect of this geometry was the catchment area at the toe of the slope which was again a minimum of 30 feet wide.

We recommended that the faces between benches be inclined approximately 1V:0.4H, rather than blasting to a 1V:1H face. In our experience, drillers have found it easier to maintain parallel hole alignments on the steeper faces. Additionally, shots with steeper holes were generally less likely to have fly rock problems. To further reduce the risk of flyrock, we recommended that a minimum of 7 feet of burden be maintained in front of the charged portion of any hole. We recommended that typical bench heights be maintained at approximately 20 feet. This would tend to promote smaller shots and hopefully reduce the potential for lengthy lane blockages that can occur with large volume shots. Additionally, it is easier to maintain blast hole spacing and alignment on the shorter faces.

SITE 81.2 - 81.4 - HUDSON VALLEY LOWLANDS SILTSTONE AND SHALE

Site 81.2 - 81.4 is located along the northbound lanes approximately five miles north of Thruway exit 18, New Paltz. The corresponding unremediated rock cut adjacent to the southbound lanes is not addressed in this paper.

The rock cut is approximately 1100 feet long and fairly linear in plan, oriented approximately North 15° West parallel to the Thruway. The maximum height is approximately 80 feet. Generally, the rock cut was very steep (near-vertical) from the ditch level upward until it intersected a sloping exposed bedding plane at which point the cut continued upward step-wise along progressive bedding planes until intersecting the overlying soil overburden slope. The dip slopes were typically covered with loose angular gravel-and boulder-sized fragments of weathered rock, i.e. scree.

The original catchment ditch ranged from about 6 to 18 feet wide and about 2 to 6 feet deep. The ditch contained abundant talus along most of its length. Most talus was gravel-sized splinters, but many, cubic yard-sized blocks were also observed, one of which had apparently fragmented into numerous small pieces upon impact. The ditch was bordered on the roadway side by a 15-foot high chain-link fence and a 4-foot high precast concrete "Jersey" type barrier at the edge of pavement.

The right-of-way line is located about 60 feet (southern end) to 120 feet (northern end) from the edge of the roadway pavement. The line was typically along level ground above the rock cut, but in one area in the central part of the cut, it crossed a 25° to 30° sloping bedding plane. The hillside above the cut was generally a gently sloped to nearly level area sparsely vegetated by grass, brush and small trees less than five inches in diameter. Soil cover was thin and in some areas bedrock was exposed. Several deep, four to five inch wide open joints and linear depressions of uncertain age were observed 10

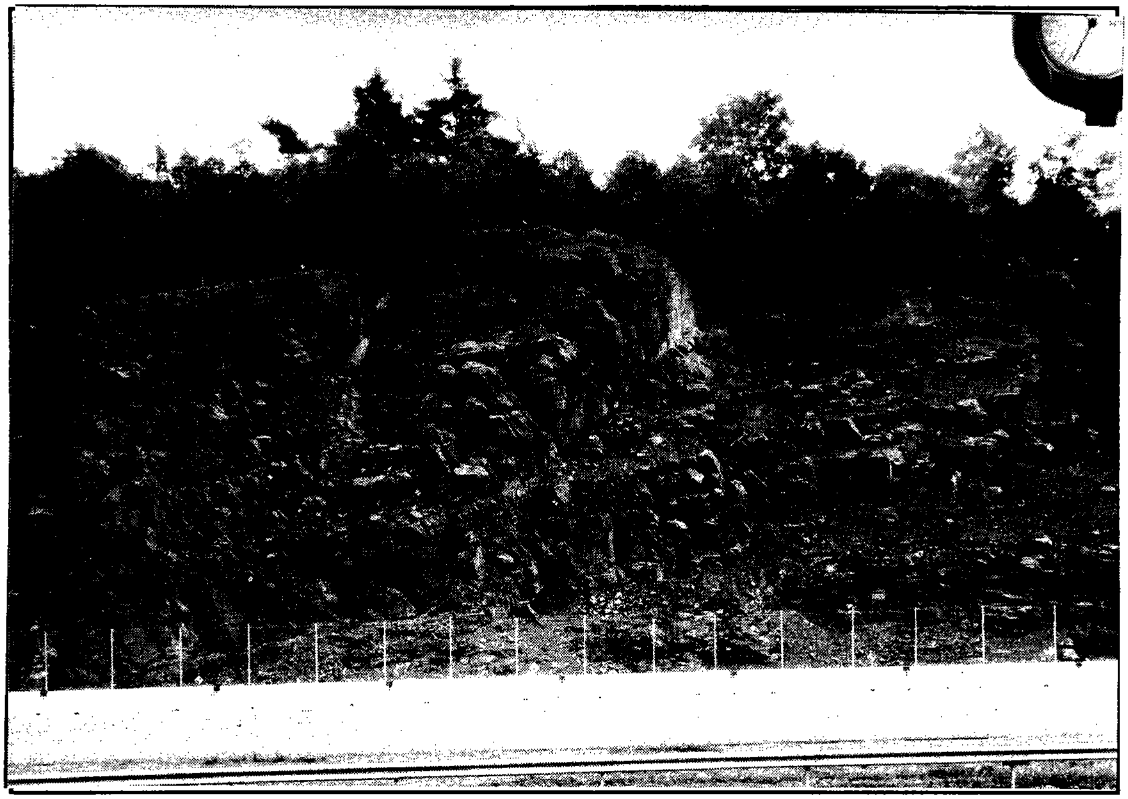

Figure 6: Photograph at milepost 81.2-81.4 showing numerous loosened and detached blocks; view looking east.

to 20 feet behind and parallel to the rock cut. The depressions were on the order of a few inches deep, one to two feet wide, and approximately 20 feet long. These depressions and joints may be related to block movement during the original blasting at this cut or to creep.

The bedrock exposed in the rock cut is predominantly interbedded siltstone and shale with some interbedded fine grained sandstone. These strata have been mapped at different times as the Snake Hill, Austin Glen and Bushkill units (Waines et al, 1983). Although the sandstone and siltstone beds are more resistant to weathering than the weaker shales, the entire rock mass is considered relatively non-durable. The surficial rock is commonly friable. Over a period of time, the exposed rock mass will continue to respond to the stresses induced by freeze thaw cycles, alternate wetting and drying, thermal expansion and contraction and other weathering processes. The weak shales will progressively slake and ravel along pre-existing weaknesses including slaty cleavage planes.

The average strike of bedding is approximately North 15° East with dips averaging about 25°, and ranging from 20° to 31° to the northwest toward the roadway. The dips increase toward the south end of the rock cut.

An orthogonal fracture pattern is well defined at the site. Some of the fractures are oxidized or pyritized; chemical weathering of the pyrite may contribute to rock mass degradation. One dominant joint set, which appears to reflect regional trends, has an average strike of North 15° East with a 70° +/- dip southeast. These steep closely spaced joints are subparallel to the rock face and occasionally form overhanging faces on the rock cut. They tend to produce detached tall, thin blocks and walls of rock. When thin enough, they have a tendency to rotate and topple from the slope. Numerous blocks appear to have fallen in this manner. In one area near the central part of the cut, several blocks about mid-height on the slope already had been bolted into place as a precaution. The orthogonal fracture set strikes approximately North 70° West, dips very steeply, and helps isolate large blocks. These fractures may be bedding slip faults.

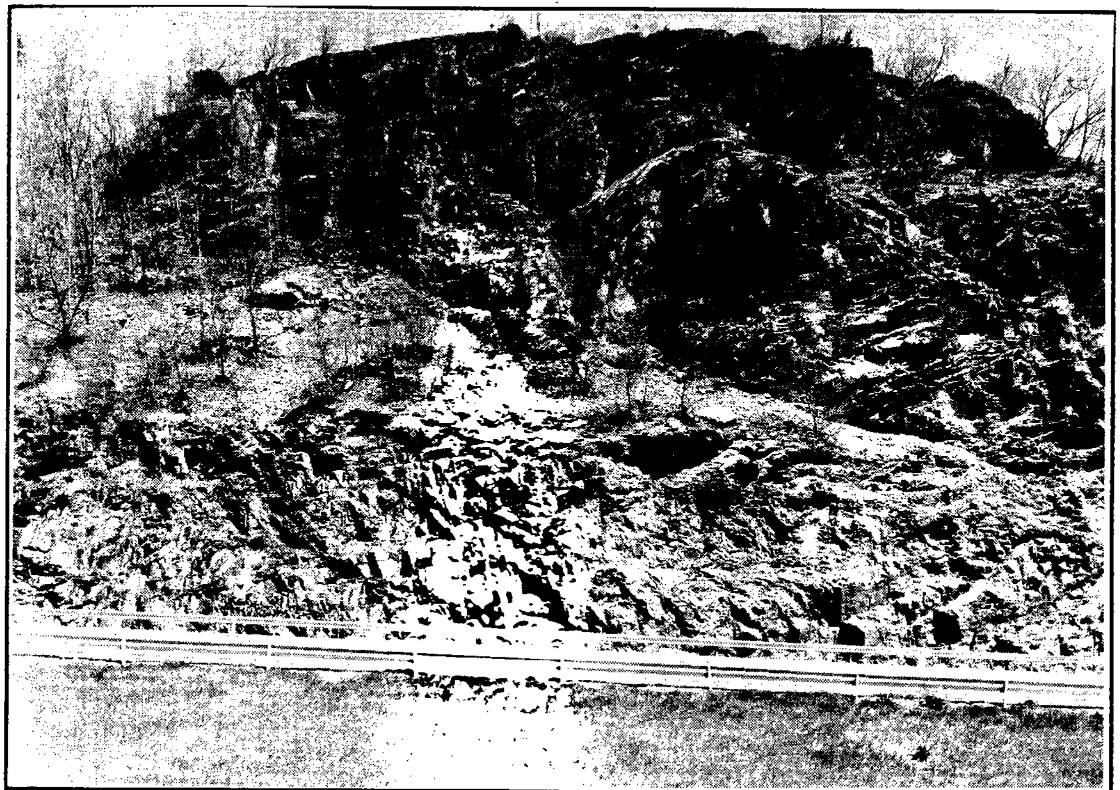

Figure 7: Photograph at milepost 34.1-34.3 showing highly fractured gneiss, steep talus slope, and an upper overlying slope; view looking east.

Many blocks up to several cubic yards in volume appeared to be in danger of failing by toppling or sliding on the rock slope (Figure 6). It was estimated that toppling failures ranging up to four cubic yards in volume might potentially occur. It was likely that the original ditch and fence system would prevent these fragments from reaching the roadway. Of more concern was the potential threat of a large translational sliding failure along weathered and/or water-lubricated shale beds with low friction angles, especially at the slightly steeper dipping southern end of the cut.

Remedial Alternatives at Milepost 81.2 - 81.4

Because of the inherent weakness of the rock at this site, measures designed to hold the rock in place would have been extensive and expensive. If the slope was cleaned, shotcrete could have been used to shield the rock from weathering and to wedge blocks together. Wire mesh could effectively guide smaller blocks to fall close to the face. Large blocks could have been bolted into place. Rockbolting had been successfully used at this slope. For protection against a deeper plane failure, long bolts could have been extended past the possible failure planes. A possible drawback to rockbolting at this site was that the rock is not very durable. It is possible that the rock could decay around and between the bolts.

Recommended Alternative at Milepost 81.2 - 81.4

The rock removal alternative for providing a catchment area had several attractive aspects. First, most of the loose weathered rock would be removed to expose relatively fresh rock. Second, by moving the slope away from the road, a zone would be created where rock could fall without endangering traffic. Third, the removed rock could be used as fill adjacent to steep embankment sections of the Thruway.

The U.S. Department of Transportation manual Rock Slopes presents a chart correlating slope height and angle to a recommended catchment width and depth to trap falling blocks of rock. This design, however, does not provide protection in the event of a large slide. To provide additional catchment volume for a large slide, the recommended catchment width was extended so that the

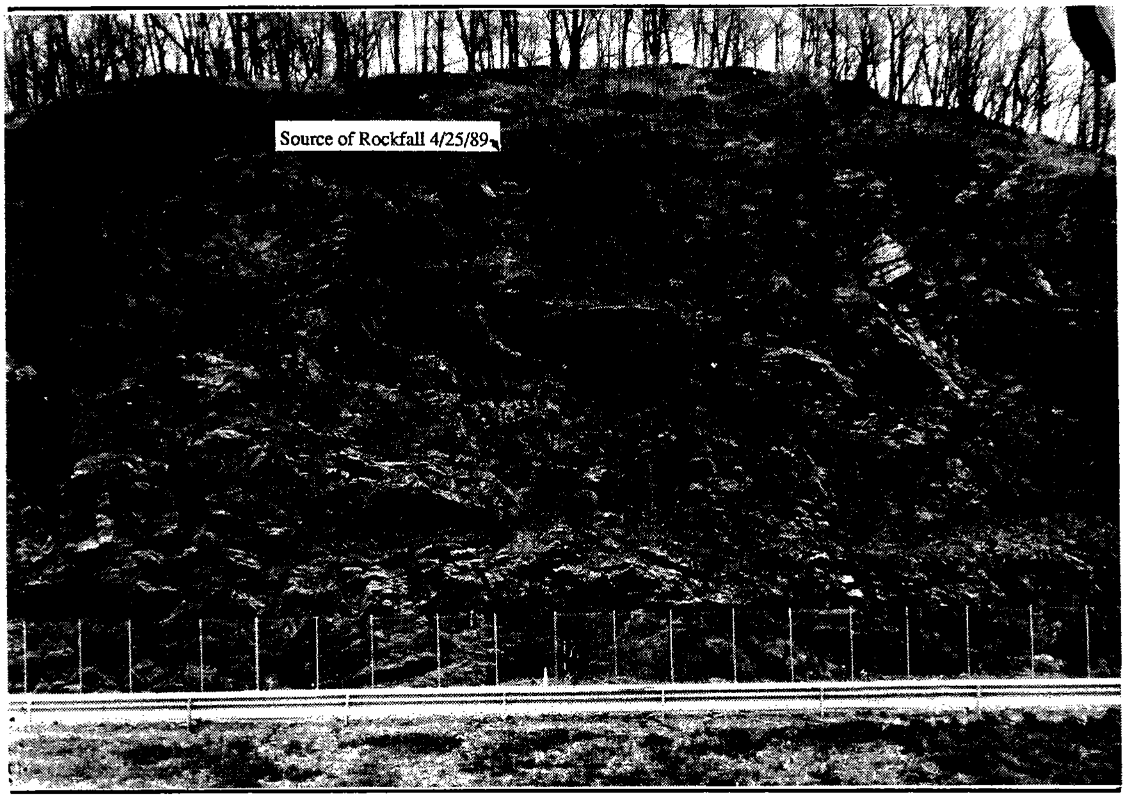

Figure 8: Photograph at milepost 41.4-41.8 showing highly fractured gneiss and source location for a recent rockfall; view looking east.

toe of the slope was to be excavated back at least 35 feet from the edge of the travelled roadway.

SITE 34.1 - 41.8 - HUDSON HIGHLANDS GNEISSES

Site 34.1 - 41.8 actually contains a group of adjacent near-vertical rock cuts, which because of their geological similarity are being addressed collectively. They are all located adjacent to the northbound lanes between the Sloatsburg Service Area and Harriman toll booths in the mountainous terrain of the Hudson Highlands. Traffic is heavy along this section of the Thruway.

All cuts are oriented north-south (+/-15 degrees). They range from 600 to 2300 feet long and 65 to 125 feet high. In many cases, the slopes continue upward beyond the rock cuts. At milepost 34.1 - 34.3, a second higher rock cut above and beyond the Thruway right-of-way towers over the lower 125-foot high cut investigated (Figure 7). The typically irregular faces contained scattered pockets of soil, loose rock and talus and were occasionally crossed by one or more benches.

Original catchment ditches ranged from 3 to 15 feet in width and were commonly one to two feet deep. The quantity of talus accumulating in the ditches was variable ranging from minor to uniformly about 1.5 feet deep at milepost 41.7 - 41.8. Most of the talus consisted of one to two cubic foot-size fragments or smaller. Occasional pieces were seen at the edge of pavement outside the ditches confirming the potential hazard to passing motorists. The only rock cut with a barrier other than a guard rail was 41.7 - 41.8, which had a 10-foot high chain link fence at the roadside edge of the ditch.

One of the critical circumstances affecting the remediation of these rock cuts was the limited right-of-way available near the crests of the rock cuts and the impracticably time-consuming process of procuring additional property. At most locations the Thruway's property line was on steep terrain close to the crests. The contiguous property was State Park land and would be difficult to purchase.

The hillsides above the rock cuts ranged from level to very steep. At mileposts 34.1 and 41.7, inclinations of the upper slopes were 30° to 40° . These slopes were covered with talus residing at a marginally stable angle of repose. Many boulders were seen accumulating at the top edge of the cut, some apparently supported in part by vegetation. The talus at 34.1 in particular, was easily dislodged during the investigation and occasionally rolled down onto the roadway despite precautions to prevent this. Spotters at the slope toe were necessary to help time these excursions with transient windows in the passing traffic and to promptly clear the roadway of any fallen debris. The steep slopes also promoted erosion of the stony soil overburden along shallow defined channels. Small talus piles accumulated in the catchment ditch below these rills and chutes. Tree trunks up to 18-inches in diameter and brush helped stabilize the debris on these upper slopes.

The exposed bedrock is typically granitic gneiss. The gneiss is occasionally schistose or cut by igneous intrusives. The rock face had a shattered, rough and uneven appearance resulting from the combined effects of dense irregular jointing, folding, faulting, blast damage, physical weathering and root wedging. Degradation of the rock face by weathering is more severe high up on the cuts; these relatively inaccessible areas appear to be the source for most of the observed rockfall debris. A rockfall that occurred at milepost 41.7 on April 25, 1989, originated near the crest (Figure 8). The complex network of intersecting, frequently closely-spaced structural discontinuities provide an opportunity for multiple modes of rock failure. It was noted that the patterns and attitudes of discontinuities observed at the accessible lower levels of the rock cuts frequently differed from those observed by lift bucket higher up on the rock faces.

The principal stability concerns at these sites were rockfalls originating from localized zones of severely fractured partially detached and loose rock, wedge and toppling failures from the rock face, and debris slides and erosion of the soil overburden from the upper slopes. The rock cuts appearing to require the most prompt attention were 41.7 - 41.8 and 34.1 - 34.3.

Remedial Alternatives at Milepost 34.1 - 41.8

The surest means of preventing rock from falling on Thruway pavement was to excavate the slopes away from the roadway. With adequate space, any rock that fell from the new face would come to rest well short of the pavement. If reliability were the only consideration, slope removal would have been the clear choice for these sites. Several other considerations governed. Two of the most significant were legal limitations as they relate to property ownership and time limitations.

Each of the slopes at the four southern sites, mileposts 34.1, 36.2, 36.8 and 41.4, have very little right-of-way space at the crest of the rock face. It would therefore have been very difficult or impossible to cut back the slopes without seriously altering land beyond the property line. The obvious solution to this problem would have been to obtain additional right-of-way as needed. The acquisition process would have been quite time consuming. The majority of the property on the east side of the Thruway is State Park land. It is likely that public pressure could have delayed (if not completely prevented) acquisition of right-of-way for as much as five years. Based on the observed condition of the rock slopes, it would not have been appropriate to delay treatment for an extended period of time. Safety of the Thruway patrons had to be addressed in a timely fashion. Slope removal was therefore not recommended as the general solution of the four southern sites.

As previously discussed, even a thorough scaling would be a temporary solution due to continuing weathering. Another disadvantage of scaling is that it is a time consuming process that can be dangerous to both the workers who are scaling the slopes and the people at the base of the slope. Scaling at the four southern sites would generally have involved a steady rain of rocks over a period of months. With the large amount of rock that needed to be scaled, it was expected that a significant number of rocks would fall into the northbound lanes. It would therefore have been necessary to ensure that no traffic was using the north-bound lanes during the times that scaling was being performed. Because of the disadvantages discussed above, scaling was not considered appropriate as a general solution to the rockfall problems. It was however, considered an appropriate supplemental measure for limited areas.

The application of systematic rockbolts, wire mesh and shotcrete was also not considered feasible. The rock faces at these southern sites are very irregular. Any attempt to fasten wire mesh to the slope would have left regular gaps of several feet between the rock and the mesh. With the vast areas that would need covering, the shotcreting would have taken a long time to complete. In addition, the logistics would have been very difficult. The slopes are high to very high. Good shotcreting practice requires the operator to move back and forth maintaining the stream of shotcrete perpendicular to the rock face. It would have been very difficult to provide safe platforms at the heights involved.

As discussed previously, in situations where it is not practical to prevent rock falls, provision is frequently made to provide a space, referred to as the catchment areas, for the rocks to fall into. This area is usually at the base of the slope and may be a wide ditch with a depth sufficient to prevent rocks from rolling out after they hit. In instances where sufficient space cannot be developed between the bottom of the slope and the areas requiring protection, it may be possible to build a structure to create a catchment area at some elevation above the bottom of the slope. If the structure rises nearly vertically from the area to be protected and the rock face is slightly inclined, the width of the catchment area will increase with height. These conditions existed at 34.1 - 41.8. The required width and height of the catchment area were iteratively determined by the height and slope of the rock face above a trial catchment height. The required width was based on the Federal Highway Administration manual.

Recommended Alternative at Milepost 34.1 - 41.8

The design of the recommended catchment structure for selected rock cuts in the 34.1 - 41.8 section was intended to satisfy several design and construction conditions.

1. The structure should be capable of being founded in an area with very little base width.

2. The system should be capable of rapid erection from a limited staging area. This favored a modular design.

3. The system should have a high degree of structural stability.

4. The components should be durable enough to perform satisfactorily under anticipated conditions for a service life of twenty years.

A system meeting these criteria was developed as a joint effort between the Fort Miller Company, the Thruway Authority, Charles H. Sells, Inc. and Dunn Geoscience Engineering Company. The conceptual system was similar to a wall of oversized cinder blocks. As conceptually designed, each block is 2'-6" wide and 4' high. Block lengths alternated between 15'-6" and 8'-6". After a stable concrete base had been poured for the first course of blocks, their narrow width would permit the wall to be erected in an area with very little base width. The catchment area would be created by filling the space between the wall and the rock face with crushed stone. Because the blocks were to be plant pre-fabricated and trucked to the site, the erection could proceed quickly with minimal staging and construction area requirements at the site.

Because the wall design is thin with respect to its height, it requires reinforcement to prevent bending and a tie-back system to prevent overturning. The conceptual tie-back system includes resin-encapsulated rock bolts drilled sufficiently into the slope to provide stable anchorage. The continuously threaded rock bolts couple to similar continuously threaded, resin coated bars which are connected to H-pile sections at 12 foot centers in the wall. The piles are anchored within the base of the wall and extend vertically through the holes in the "oversized cinder blocks". As the wall is erected, the holes containing the H-piles are filled with concrete. This encases the piles and the tie-back connections to limit the potential for corrosion.

As the wall is backfilled with freely draining crushed stone, there is the possibility that the protective resin coating of the tie bars could be cut permitting corrosion to begin. For this reason, each tie-back would be provided with a 4" diameter plastic pipe sleeve.

To provide a catchment trap and reduce the required wall height, a rock catchment fence is included along the top edge of the wall. The catchment width is defined as the horizontal distance from the top of the rock catchment fence to the average surface of the rock face. The fence posts are founded in the fill so that, if the fence is damaged by a major fall, the wall may escape damage.

Brugg Cable Products, Inc., of Sante Fe, New Mexico (U.S. corporate office and fabrication plant) is the sole supplier of the cable mesh fence. From 1955 and to 1965, the Swiss Federal Institute of Snow and Avalanche Research was actively involved with the Brugg Company on providing rockfall and avalanche observation data to assist in designing a system to withstand those types of naturally occurring conditions. The cable mesh fence has been used widely throughout Europe since 1968, and in the U.S. since 1985 without serious problems reported regarding the fence's effectiveness. For full details of the system as constructed, the reader is directed to reports prepared by the Thruway Authority or to the presentation at this symposium by R. Cross.

SUMMARY AND CONCLUSIONS

1. Rock slopes present the greatest risk to the public when there are high speed, large traffic volume highways immediately below them.

2. The greatest threat is typically posed by weathered rock close to the top of the slope. This rock is likely to be the least stable and least accessible to examine, and possesses greater potential energy to fall, slide and roll farther from the toe.

3. Reliance should not be placed on conditions that are apparent near the base of a tall cut slope. The rock at the toe of a cut was probably unweathered when the highway was excavated. Structural domains may differ at the crest and toe of tall slopes. A high-reach lift bucket may be the only means of closely inspecting the worst conditions on the slope.

4. The difficulty of adequately inspecting and maintaining high cuts underscores the importance of using reliable, long-term options to reduce the risk of rockfall damage.

5. In most circumstances, weathering will deteriorate the exposed face of a rock slope long before the useful life of the highway is passed.

6. The most reliable method of providing for the safety of the highway public is to accept that slope deterioration will occur and to provide a safe place for rocks to fall.

7. In situations where it is not possible to provide an adequate catchment area at the toe, consideration should be given to providing a catchment zone at some height above the roadway. This zone can be established by a wall or by the top of an adequately designed rock fence.

8. In addition to the use of adequately designed catchment areas, the cuts should still be properly blasted, scaled, rock bolted, shotcreted and/or meshed to reduce the amount of rockfall.

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Rock Slope Inventory, Evaluation and Remediation for Sections Along the New York State Thruway

Joseph S. Burke, PE
Stephen LeFevre, CPG

Abstract

Recent attention to the aging highway infrastructure has concentrated on various aspects of safety and reliability. With some notable rock falls along the New York State Thruway, attention has been drawn to rock slope stability, rock fall mitigation and rock slope remediation. Rock slopes established during construction of the main thrust of the interstate highway currently represent a challenge to the geologic and engineering community to accurately evaluate rock slope safety. The goal of this investigation is to inventory rock slopes, evaluate stability, prioritize areas to be remediated, and to design remedial measures. The proposed remediation and mitigation must conform to the constraints of the existing highway right of way, allow for maintenance of traffic, and represent the most cost effective solution to rock slope weathering and rock fall occurrences.

The approach taken when faced with rock falls or rock slope stability evaluation involves implementation of a set of procedures to:

1. inventory all rock slopes
2. evaluate rock fall hazard
3. prioritize based upon a rating system
4. remediate using various mitigation techniques.

The procedures formulated for inventorying and evaluating rock slopes, for 230 rock outcrops along the New York State Thruway north of Nyack, N.Y., consisted of:

1. evaluation of available methods
2. compilation of existing rock fall data
3. remediation alternates
4. design requirements.

Remedial action plans were developed for 35 high hazard slopes identified by the inventory as having the highest priority. Remediation included rock removal, use of rock catchments, rock fence, guide rail, and steel mesh on rock slopes.

Introduction

The New York State Thruway was constructed during the 1950's to provide highway access between major New York State cities. The highway was built as a toll facility under the control of the New York State Thruway Authority (NYSTA).

The highway had experienced rock falls and rock slope failures previously however, after a notable rock fall, the New York State Thruway Authority retained Clough, Harbour & Associates (CHA) to conduct a field survey of the geologic conditions of rock cut slopes along the New York State Thruway from mile post 19.0 ± to mile post 215.0 ±. This involved some 230 rock slopes. Included as part of the work was establishment of a procedure to rank slopes according to hazard classification and finally to design remedial measures for the most hazardous slopes.

Location

The main body of the highway begins just north of the New York City line (milepost 0.0) and runs north to Albany (milepost 141.9). A portion of the highway extends eastward at Exit 21A (milepost 133.6) to the Massachusetts border, this section is referred to as the Berkshire section. At Albany the highway changes direction and extends westward to the cities of Utica (milepost 232.8), Syracuse (milepost 276.5), Rochester (milepost 350.9) and Buffalo (mile post 421.5). The New York State Thruway encompasses other portions such as the Garden State Parkway Connection, the New England Section (I-95) and the Niagara Section (I-90) in Buffalo. See Figure 1.

The highway passes through three major physiographic provinces; the Hudson Highlands (mile post 13.0 ± to mile post 47.0 ±), the Catskill Mountains (mile post 55.0 ± to mile post 124.5 ±) and the Taconic Mountains (Exits B1 to Exit B3). The bulk of the rock slopes inventoried were within these areas.

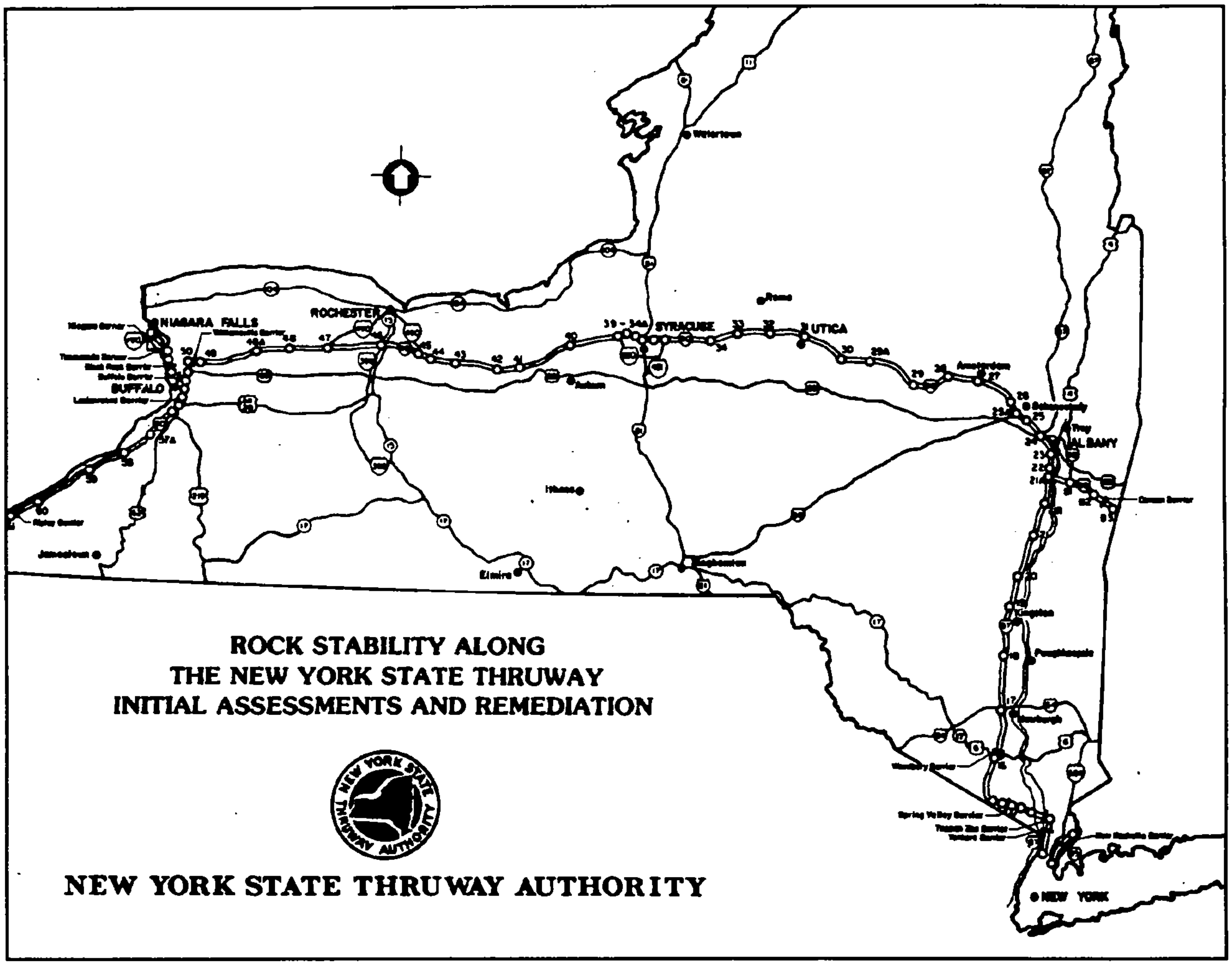


Figure 1: Project location

Methods

Prior to initiation of the field investigation, the Authors' firm reviewed the available published information for inventorying rock slopes and slope ranking systems to determine rock fall hazard. The purpose was to develop a reliable procedure to identify those slopes which posed an imminent potential for rock falls, rock slides or rock slope failures which would result in rock debris reaching the roadway surface.

The most rigorous rock slope inventory method and ranking system known to the authors' at the time was that presented by Golder Associates at the 1987 FHWA RockFall Mitigation Seminar [1]. The form had been slightly modified by the New York State Department of Transportation (NYSDOT), who were also simultaneously performing their own rock slope inventory (see table 1).

The extent of the forms modifications was to delete items such as slope length, visibility and traffic density, items which would affect the ranking and potentially skew the results for determining rock fall potential. The form developed by Golder Associates was an all inclusive probabilistic approach where, for example, inadequate containment could be offset by short cut lengths.

Two items were added to the form to include slope angle and distance to rock catchment (ditch line). It was felt that the form did not adequately address the NYSTA's concern with identifying those slopes capable of having rock falls which could reach the roadway shoulder area. For example, there was the potential that a rock slope

Table I: Typical Rock Slope Inventory form (Modified Golder Associates form)

Division:________________________ Section:__________

Milepost Location:______________________________

Direction of Travel:______________________ Date:__________

	Points 1	Points 3	Points 9	Points 27	Points 81
Slope Height	< 15 Ft.	15 to 25 Ft.	25 to 35 Ft.	35 to 45 Ft.	> 45 Ft.
Slope Angle					
Edge of DL to Face of Rock Distance					
Ditch Dimensions	Meets Ritchie criteria	Adequate width, inadequate depth	Moderate catchment	Limited catchment	Nil
Geology Observations	Massive, no fracture dipping out of slope	Discontinuous fractures, random orientation	Fractures form wedges	Discontinuous fractures dipping out of slope	Continuous fractures dipping out of slope
Block Size	< 6 in.	6 to 12 in.	1 to 2 Ft.	2 to 5 Ft.	> 5 Ft.
Rock Friction	Rough, irregular	Undulating	Planar	Smooth, slickensided	Clay, gauge, faulted
Water/Ice	Dry, warm winters	Moderate rainfall, warm winters	Moderate rainfall, some freezing	Moderate rainfall, cold winters	High rainfall, cold winters
Rock Fall History	No falls	Occasional minor spalls	Occasional falls	Regular falls	Major falls/slides

Recommended Mitigation Method

greater than 45 feet high and with adequate containment would rank the same as a rock slope 25 feet high with no containment.

In addition, a separate form for qualitative structural analysis of rock slopes was developed by the authors' firm, see table 2. This form was used in the field to rank slopes according to a hazard classification from "A" to "E". "A" hazard slopes had the highest risk for rock falls and "E" slopes had minimal risk of rock falls, see Table 3 for hazard priority ranking system. The modified Golder Associates form was used to develop a ranking for the high hazard "A & B" slopes.

Available Information

The NYSTA made available to the authors' firm historical information regarding rock fall occurrences and remediation. The information included maintenance records, police reports and engineering reports performed by the NYSDOT Soil Mechanics Bureau. The information was helpful in identifying areas where consistent rockfalls had occurred.

Discussion of Geology

While an in-depth discussion of the various geologic units (and their origin) associated with the New York State Thruway is considered to be beyond the scope of this paper, a cursory knowledge of their complex structural make-up and tectonic history is essential if one is to gain a proper understanding of both rock slope stability and rock slope failure within the study area.

For the purpose of describing the geology in the immediate vicinity of the Thruway from milepost 19.0± to milepost 215.0±, the authors make reference to the Roadside Geology of New York by Van Diver (1985). This guidebook provides a thorough, yet concise, summary of the geology that can be observed while travelling along the New York State Thruway.

The following is a brief description of the pertinent geological features that occur as one progresses from Suffern north to Albany and then west to Herkimer. The Berkshire Spur section of the Thruway will be discussed last.

Beginning at the southernmost extent, Suffern marks the location of the Ramapo Canopus fault, a border fault that delineates the boundary of the Hudson Highlands. The Hudson Highlands are characterized by highly fractured gneisses that display distinctive layering in some

Table II: Qualitiative Slope Evaluation Form

CLOUGH, HARBOUR & ASSOCIATES
ENGINEERS, SURVEYORS & PLANNERS

FIELD CREW:........................ ________________

____

____

CLIENT:........................ ________________

PROJECT NAME: .... ________________

OUTCROP MILE:... ________________

DATE:................. ________________

FILE NUMBER:..... ________________

QUALITATIVE STRUCTURAL ANALYSIS

1. **ROCK TYPE/LITHOLOGY:** ________________
   HARDNESS:.................. ________________
   WEATHERING:.............. ________________
   COLOR: .... ________________

2. **ROCK STRUCTURE:**

   BEDDING THICKNESS: ________________
   BEDDING DIPS: Toward/Away From NYS Thruway
   FOLIATION, S_1 DIPS: Toward/Away From NYS Thruway
   FOLIATION, S_2 DIPS: Toward/Away From NYS Thruway
   JOINTS DIP: .... ________________
   OTHER: .... ________________

3. **SEEPAGE OF GROUNDWATER CAUSING MECHANICAL WEATHERING FROM FROST ACTION:**

   DOES NOT EXIST: .... ________________
   EXISTS ALONG BEDDING: ...... ________________
   EXISTS ALONG FOLIATIONS: .. ________________
   EXISTS ALONG JOINTS: .... ________________

4. **SEEPAGE OF GROUNDWATER CAUSING CHEMICAL WEATHERING:**

   DOES NOT EXIST:........................ ________________
   EXISTS ALONG BEDDING:............. ________________
   EXISTS IN FOLIATIONS: .... ________________
   EXISTS ALONG JOINTS: .... ________________
   EXISTS ON OUTCROP SURFACE: ________________

5. **SURFACE RUNOFF THAT IS SEEPING THROUGH DISCONTINUITIES IN ROCK CAUSING MECHANICAL WEATHERING:**

   DOES NOT EXIST:........................ ________________
   EXISTS IN BEDDING:..................... ________________
   EXISTS IN FOLIATIONS: .... ________________
   EXISTS IN JOINTS: .... ________________
   EXISTS ON OUTCROP SURFACE : ________________

6. **SURFACE RUNOFF THAT IS SEEPING THROUGH DISCONTINUITIES IN ROCK CAUSING CHEMICAL WEATHERING:**

   DOES NOT EXIST:........................ ________________
   EXISTS IN BEDDING:..................... ________________
   EXISTS IN FOLIATIONS: .... ________________
   EXISTS IN JOINTS: .... ________________
   EXISTS ON OUTCROP SURFACE: ________________

7. **SLOPE CREEP:**

   NONE: .... ________________
   SLIGHT:............. ________________
   MODERATE: .... ________________
   EXTREME: .... ________________

8. **EROSION:**

   NONE: .... ________________
   SLIGHT: .... ________________
   MODERATE: ... ________________
   EXTREME: ...... ________________

9. **SOLUBILITY OF ROCK**

   NEGLIGIBLE:...... ________________
   VERY SLIGHT:.... ________________
   SLIGHT: .... ________________
   MODERATE: ...... ________________
   EXTREME: .... ________________

10. **HISTORY:**

____

OTHER COMMENTS:

____

SKETCH: (Should include height of outcrop, distance to white line & guard rail, depth of drainage swale, length of outcrop, rock outcrop features, & approximate direction of north.)

areas. According to Van Diver (1985), the New York State Thruway then passes over Paleozoic sedimentary strata between milepost 43.0$\pm$ and milepost 142.$\pm$ (Albany). This is evidenced by numerous outcrops that reveal extensive folding and faulting. Within this almost one hundred mile stretch, one can observe Cambrian-Ordovician age limestone and dolostone of the Wappinger Group, and Silurian- Devonian age congomlerate and sandstone that forms the base of the Schunemunk- Green Pond graben.

North of the Schunemunk - Green Pond graben, the bedrock floor is composed of Ordovician shales and graywackes of the Normanskill and Martinsburg Groups. The prominent Shawangunk scarp is evident to the west of the Thruway in the vicinity of New Paltz.

Outcrops occurring along the Thruway between Rosendale and Albany reveal both Devonian age limestones of the Onondaga and Helderberg groups, and shales and siltstones of the overlying Hamilton Group (Van Diver, 1985). Deformed bedding can be observed in many of the outcrops, and at some of the exposures anticlinal folds and/or faults are evident.

Throughout the stretch of roadway between Exit 21 and Exit 21B there exist extensive roadcuts that expose highly deformed limestone units of the the Onondaga and Helderberg Groups. Examination of the numerous outcrops indicate that upfolded anticlines and downfolded synclines are present, in addition to shale beds that are oriented nearly vertical (Van Diver, 1985).

Travelling north along the Thruway to Exit 24, one can observe graywackes and shales of the Austin Glen Formation which exist in the form of erosional remnants of the Taconic Klippe. Briefly, the Taconic Klippe refers to the complex distribution of north-northeast trending Cambrian and Ordovician age rock masses that apparently either slid in titanic landslides or were transported westward along great thrust faults during the Taconian mountain building event, and then isolated by erosion.

The structural make-up of the bedrock underlying the New York State Thruway for the stretch from Albany west to Herkimer generally consists of a series of block faults. Exposures of Ordovician shale occur between Exits 25 and 26 in the vicinity of John Boyd Thacher State Park, which is situated at the top of a scarp.

Heading west from Exit 26, the Thruway passes over a significant block fault at Pattersonville, which is referred to as the Hoffmans fault. The geologic units exposed in outcrops between Amsterdam and Exit 27 are comprised of Canajorie shale overlying Black River limestone.

The next outcrop of notable significance occurs between milepost 188.0± and milepost 189.0±, where Cambrian Little Falls dolostone unconformably overlies Precambrian gneiss (Van Diver, 1985).

Several transecting faults are present between Canajoharie and Little Falls. Outcrops existing within this particular fifteen mile stretch are composed of Utica shale, as well as Ordovician age limestones and dolostones of the Beekmantown, Black River, and Trenton Groups.

Table III: Hazard Priority Ranking

A Priority: Moderate probability of sufficient volume to cause accident if failure undetected.
B Priority: Some probability of failure of sufficient volume to result in accident if failure undetected.
C Priority: Moderate probability of failure of small volumes which might reach the road.
D Priority: Moderate probability of localized failures under estreme climatic conditions.
E Priority: Slight probability of localized failures under extreme climatic conditions.

Slightly folded and faulted Utica shale is also exposed in several roadcuts in the vicinity of Exit 29A. At milepost 212.8± the Little Falls fault, a major block fault, crosses the Thruway.

The remaining section of the Thruway whose geology has not yet been discussed is situated between Exit 21 and the Massachusetts border. The geology in this area is a product of the Taconic Klippe, which was previously discussed in this section. Roadcuts along the Berkshire Spur reveal significantly folded and faulted slates and phyllites. According to Van Diver (1985), the relative grade of metamorphism increases as one progresses eastward.

Field Investigation

The field investigation began in March 1988 and was completed approximately six weeks later. The field investigation involved two stages. Stage I consisted of inventorying all rock slopes within the study area, and Stage II involved collection of design data for the highest ranking slopes.

The field investigation was performed by a team composed of two thruway personnel and two CHA personnel, and included an engineer, two geologists and a technician. The field team members were kept consistent during the inventory to insure uniform interpretation of field conditions [4].

Stage I Field Investigation

The inspection included completion of a qualitative structural analysis using a form developed by CHA, and use of the modified Golder Associates slope inventory form. The field investigation focused on identifying rock fall potential based upon major categories or causes of rock fall including:

- rock structure and orientation, lithology
- mechanical weathering
- chemical weathering
- hydrostatic pressure
- blast damage

Observations were made as to the immediate potential for rock falls to occur which would adversely impact the highway. This included observations of rock debris in catchment areas, rock overhangs, blocking of rock slope face, and peeling of rock slope face. These observations were correlated to the proximity of rock slope to roadway shoulder, rock slope geometry, available rock catchment and presence of thrie beam double backed guiderail.

Stage I Field Investigation Results

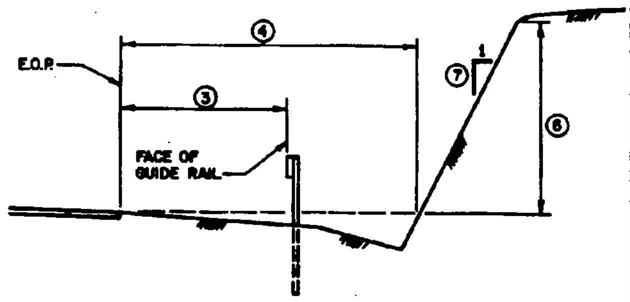
At the completion of the stage I field investigation, the field data was assembled and reduced in order to rank all slopes according to their potential for rock fall or rock slide. From this list the highest priority slopes were to be evaluated so that the most hazardous slopes could be remediated.

Stage II Field Investigation

All slopes determined to be "A & B" hazard slopes, 35 slopes in all , were inspected a second time to collect additional design remediation information. All rock slopes were delineated by highway stationing and then were photographed. Figure 2 was used to collect guiderail information, including guiderail transition areas. Table 4 was used to more accurately define rock cut dimensions, such as variation of slope height, ditch depth and width, distance from toe of slope to the roadway, and initial assessment of required treatment.

Rough use of photographic mapping was used to provide information regarding rock slope height [3]. The height of slope and slope angle data was used to check rock catchment adequacy for design of new catchments, and for quantity take off information, i.e. scaling and blasting

SECTION: ________ MAINTENANCE SUPERVISOR: ________



1. GUIDE RAIL: YES ☐ NO ☐
2. TYPE: BOX BEAM ☐ CORR. BEAM ☐ CABLE ☐
3. DISTANCE FROM E.O.P. TO FACE OF GUIDE RAIL ________ FT.
4. DISTANCE FROM E.O.P. TO FACE OF ROCK ________ FT.
5. LENGTH OF ROCK OUTCROP ________ FT.
6. HEIGHT OF ROCK: MIN. ____ FT.; MAX. ____ FT.
7. SLOPE OF ROCK FACE: ONE ON ____ .

PHOTOGRAPHS: ROLL No. ___ PICTURE Nos. ________

HISTORY:

Figure 2: Slope Inventory Form - Design Information Sheet

quantities, mesh area, etc.

Stage II Field Investigation Results

At the completion of the Stage II field investigation, a final ranking of the slopes was performed using the modified Golder form to establish a numerical priority rating for each of the slopes evaluated during the Stage II Investigation. The highest priority slopes were to be remediated first.

Remediation Methods

Remediation plans were developed for the highest priority slopes. The plans called for the slopes to be let under three separate contracts grouped by geographic location.

The proposed remediation consisted of the following types of remedial measures:

- rock removal by scaling or blasting
- rock catchment improvements

Table IV: Rock Slope Inventory Form

FIELD CREW________________ ROCK CUT MILE POST LIMITS________________

CLIENT________________ DATE________________

REFERENCE MILE POST________________ PROJECT #________________
(= 10 + 00)

ROCK CUT DIMENSIONS

STATION	HEIGHT	HEIGHT OF CUT RANGE	DEPTH OF CUT RANGE BOTTOM TO TOP	NETTING, SCALING, DRAINAGE AND OTHER COMMENTS

- rock mesh
- rock fence

The basis for choosing which method to use consisted of judging which method or combination of methods reduced the rock fall hazard in a cost effective manner.

The initial remediation treatment called for removing all unstable rock masses noted during the inspection which would result in toppling failures, see figure 3,plane and wedge failures, see figure 4, and areas where rock weathering or jointing was causing rock falls. The method of rock removal was by scaling, blasting, or removal by mechanical methods, to slope geometries determined to be stable. Rock removal measures were typically the most costly. The rock removal limits were delineated using a typical detail and associated table indicating design removal limits, see figure 5 and table 5. The rock removal limits were dictated by a determination of the necessary rock removal that would provide a stable slope, or conversely to geometries where potential rock falls could be dealt with using the other referenced methods. Constant problems encountered with rock removal measures were limited highway right of way, treatment of earthen slopes overlying the removed rock mass, construction procedures for achieving the rock removal, and maintenance and protection of traffic.

The other methods used to remediate rock slopes were designed to prevent future occurrences of rock falls from reaching the roadway surface and included use of rock catchments. Rock catchments were proposed to capture any particles falling from the rock slope face. At most rock slope locations rock catchments already existed, although these catchments were installed primarily as ditches for highway drainage. In all remediation areas available rock catchment was evaluated based upon the FHWA recommended ditch design chart, see figure 6. The design of catchments by use of this diagram was based on catchment width and depth dependent upon slope height and slope angle. If the catchment was inadequate, an evaluation was made as to the feasibility of improving the catchment up to the recommended standards set forth by FHWA. An 18 inch sand cushion was placed in the bottom of the ditch to prevent rock particle deflection out of the containment area. Two types of rock catchments were used; "Type A" catchments involved cutting a trench to provide adequate catchment, see figure 7.

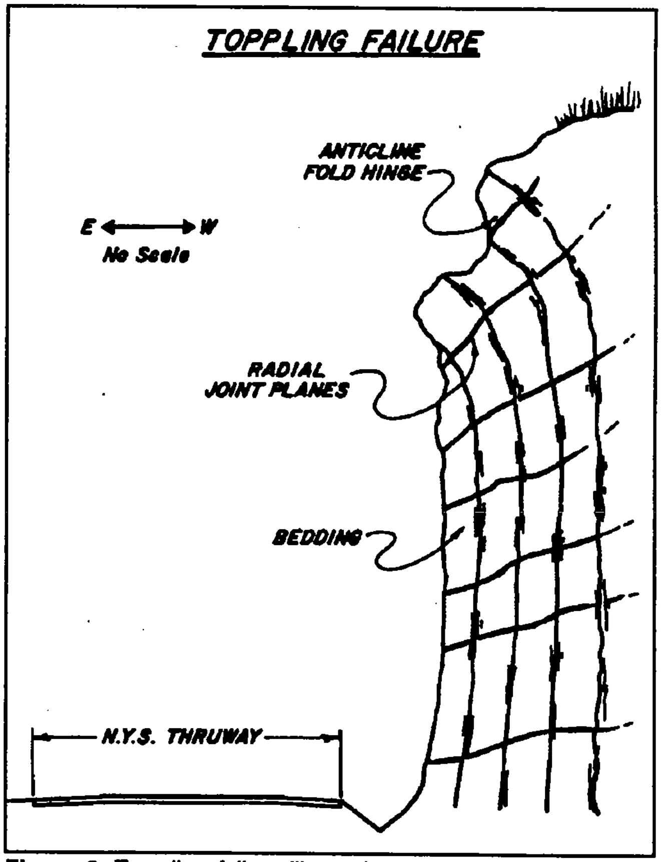


Figure 3: Toppling failure illustration

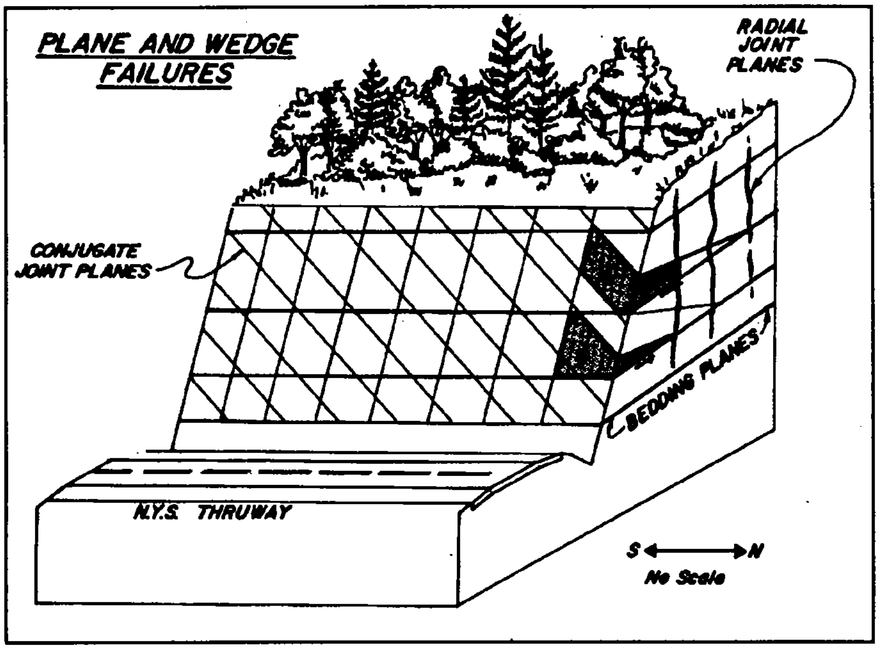


Figure 4: Plane & wedge failures illustration

Table V: Rock Removal Schedule

TABLE OF ROCK SCALING

Location	M.P.	Station	Length	H_1	H_2	H_3	C_0	C_1	C_2	C_3	Scale	Blast	Item 25203.3205 C.Y.	Item 25203.3206 C.Y.	Remarks
15NM	102.40	2+00													
			100	0	15		2		4		X	X	454	113	See Note 3
		3+00													
			300	0	20		2		5		X	X	2,117	529	
		6+00													
			550	0	20		1		5		X	X	3,326	832	
		11+50													
			100	0	0			0	0						
		12+50													
			100	0	20			3	4		X	X	706	176	See Note 3
		13+50													
			250	0	30			2	4		X	X	2,268	567	
		16+00													
			100	0	20			0	3		X		378		
		17+00													
16SB	102.52	0+00													
			150	4	15			0	4		X	X	333	83	See Note 3
		1+50													
			900	0	15		1		2		X		2,552		
		10+50													
			300	4	35			2	5		X	X	3,069	767	See Note 3
		13+50													
			220	4	35			1	4		X	X	1,608	402	
		15+70													
			230	4	20			1	4		X	X	927	232	
		18+00													
			100	2	12			1	4		X	X	252	63	
		19+00													

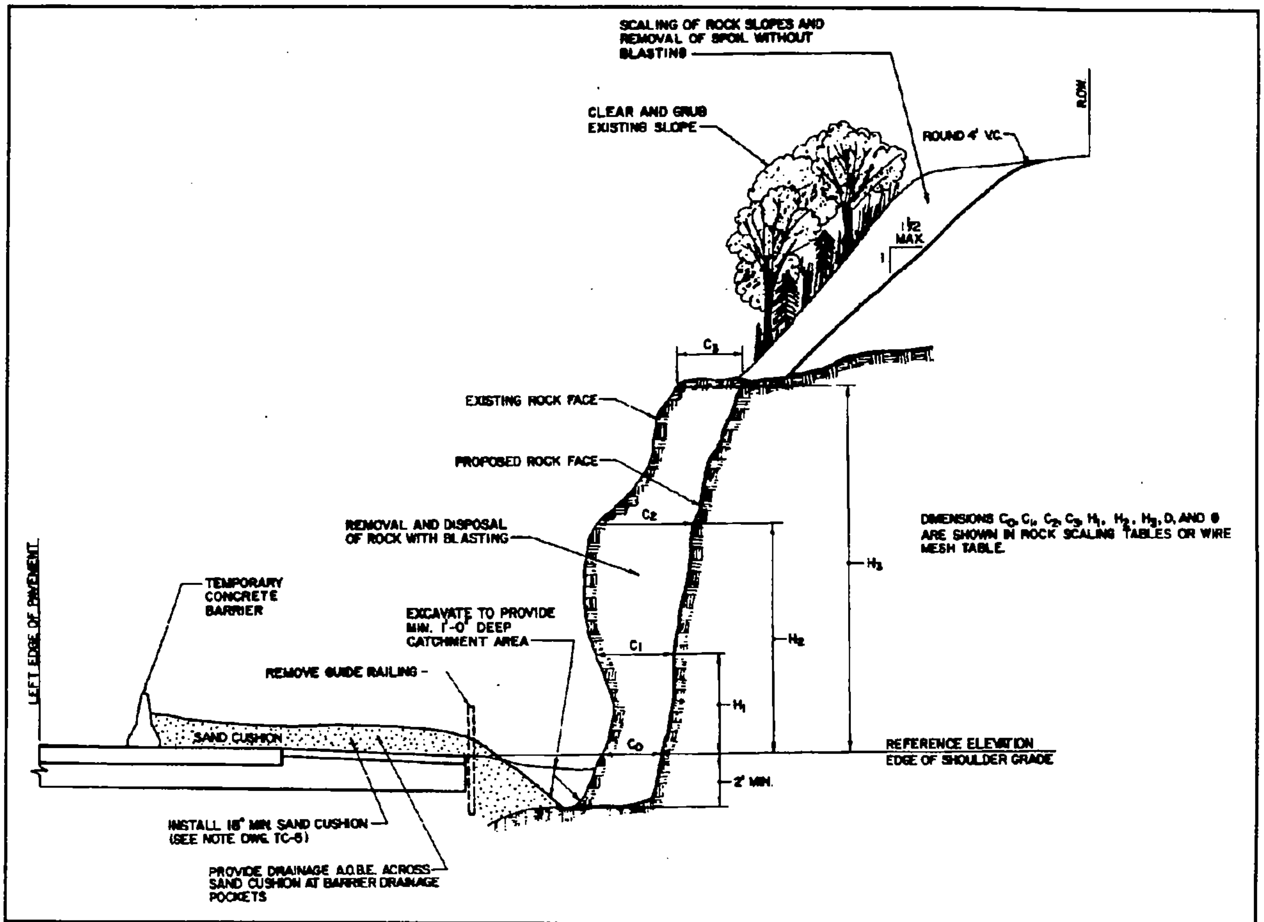


Figure 5: Rock removal, typical detail

This type of catchment was typically used in areas where distance between the roadway and the toe of slope was limited, and the catchment also functioned as a drainage ditch. At some locations where ditch depth had to be limited, corrugated block-out thrie beam guiderail was used to increase catchment depth, see figure 8. "Type B" catchments involved construction of earthen berms to increase effective ditch depth and width. This type of catchment was used where the distance between the roadway and toe of slope was greater than the nominal area required for a cut catchment, and where the catchment was not used for highway drainage. The attractiveness of using this alternate was that it avoided a typically required rock cut for construction of the catchment, see figure 9.

At many locations practical restrictions, such as available distance between the roadway shoulder and the toe of the rock slope, made it very difficult to meet the ditch design criteria for rock catchment.

If catchments could not be brought up to the recommended width and depth, alternate means of preventing loose particles from reaching the roadway were implemented. These methods included, among others placement of rock mesh across the rock face. Rock mesh was used in those areas where the nature and extent of the rock weathering or jointing resulted in a considerable number of rock particles falling. These areas typically had a sound rock mass and inadequate rock catchment. Removal of the rock slope to achieve the required containment capacity was not practical or cost effective. The rock mesh was loosely draped across the rock slope. The mesh was held in place at the top of slope using resin rock bolts. Additionally, the mesh was held along the slope face using grouted steel bars which allowed the mesh to freely separate from the slope so that particles could fall to the toe of slope within the confines of the mesh, see figure 10.

The last method employed the use of a rock fence [2]. The rock fence, like the rock mesh, was used in those areas where the nature of rock weathering or jointing resulted in a considerable number of rock particles

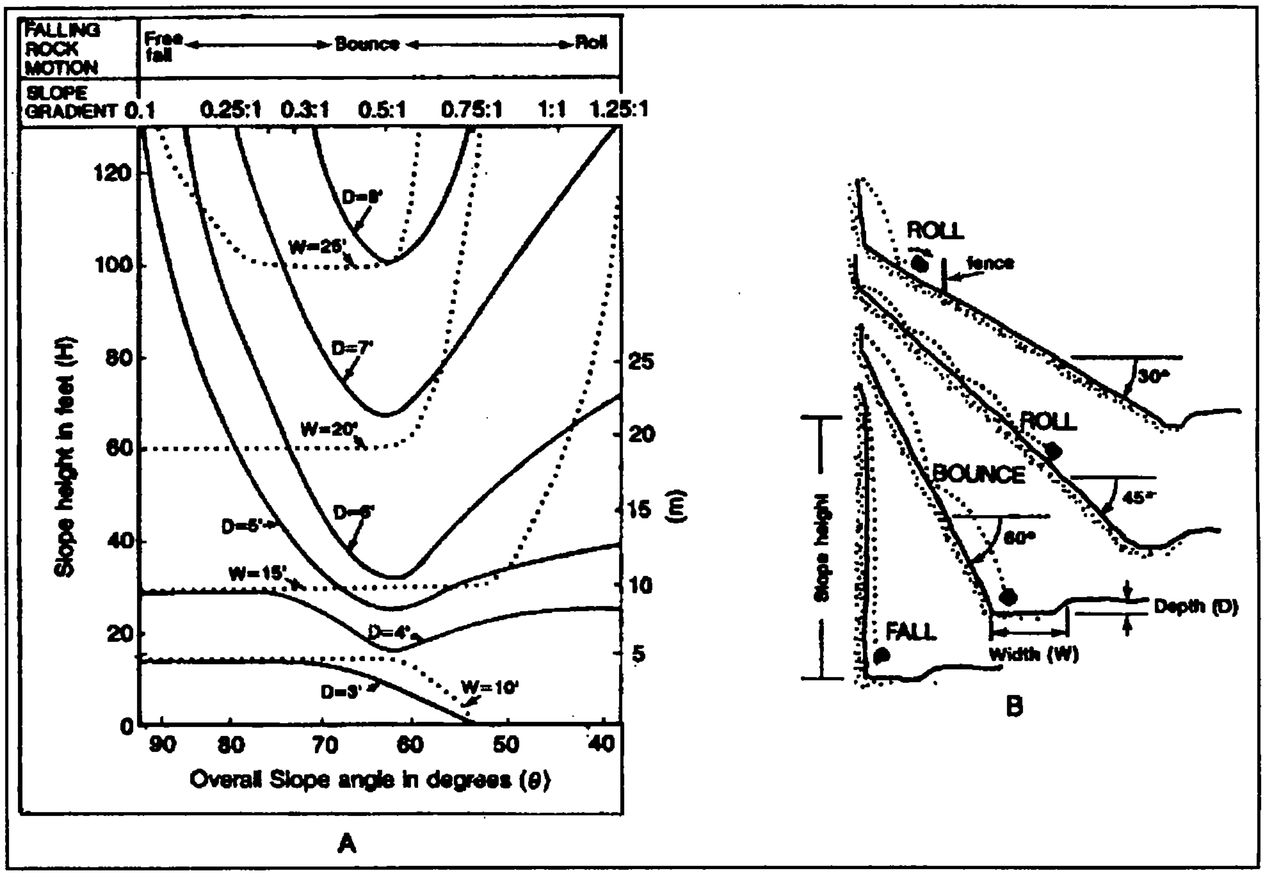


Figure 6: Ditch Design Chart

falling. Again, these areas had stable rock masses or stabilized rock masses and inadequate rock catchment.

The difference in utilizing a rock fence as opposed to rock mesh was rock fence was generally used for slopes of lower height, typically slopes less than 25 feet in height where larger diameter particles might be expected. However, the use of the fence was dictated not only by the slope height, but also by slope angle, rock particle type and size, and slope characteristics such as presence of ledges and weak zones. Rock fence was used to prevent potential free falling particles from reaching the roadway, see figure 11.

Use of large diameter rock bolts were not specifically called for at any site. However, they were available as an alternate method if unstable rock masses which could not be removed safely or cost effectively were encountered during the remediation program. Use of rock bolts to hold unstable rock masses in place were not considered as an acceptable primary method of treatment. Use of rock bolts, in this application, was looked upon as a method of last resort. A previous rock slide involving a bolted section of rock made this option undesirable.

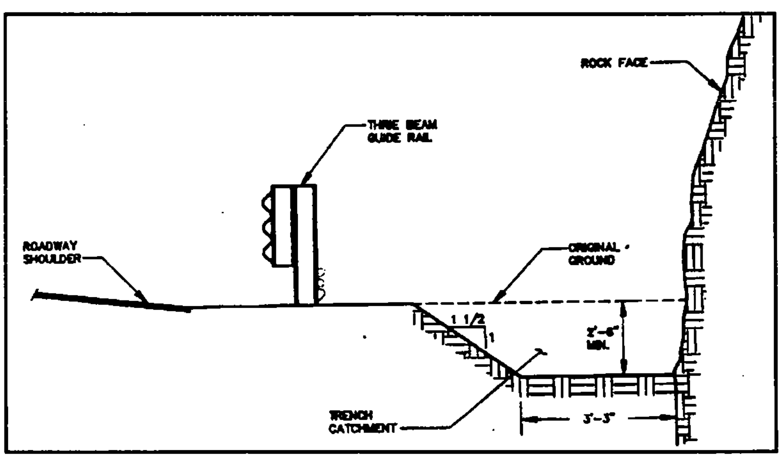


Figure 7: Catchment Type A; typical section

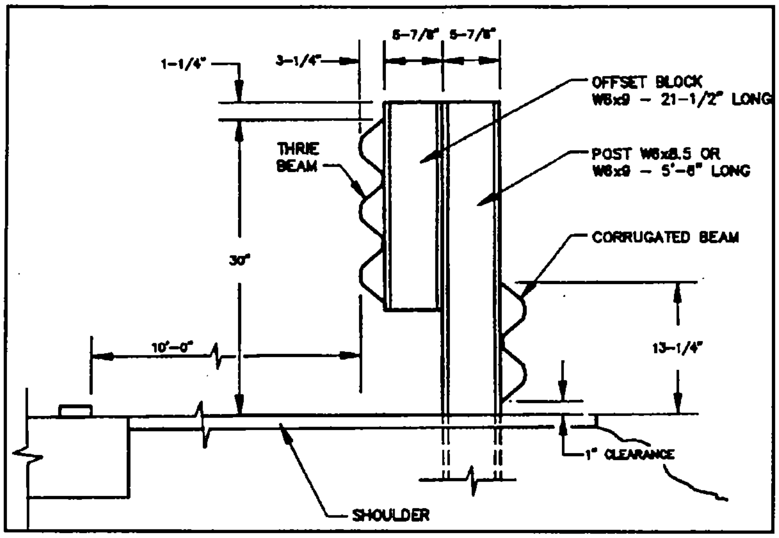


Figure 8: Typical section for corrugated block-out thrie beam.

Conclusions

The rock slope inventory, evaluation and remediation that was performed for sections along the New York State Thruway Authority involved development of a procedure for evaluating rock fall hazards in the field, reduction of the data to a prioritized ranking, and design of remedial measures for high priority slopes. The method for field investigation involved application of a qualitative structural analysis using a form developed by the authors' firm in conjunction with a modification to an existing ranking system developed by G Golder Associates. The slope hazard was determined qualitatively ranging from "A", high risk, to "E", minimal risk. The high risk slopes were inspected again for reevaluation using a point rating system in order to gain remediation design data. The "A" hazard slopes were subsequently remediated using various methods to reduce immediate potential rock fall hazards, and to reduce the future occurrence of rock falls and further reduce the probability of rock falls reaching the roadway surface. The remediation methods utilized included; rock removal by scaling, blasting or other mechanical means, construction of rock catchments, use of rock mesh and use of rock fence.

The field investigation and remedial design work described in this paper represents a very rapid response to a very complex problem. The employed methods for evaluation of rock fall hazards did not involve in depth slope stability evaluation, mapping or surveying techniques. Remedial measures were direct and focused. They were conceptually simple, low in technology and rapid in application. Construction remediation began within several months of the initiation of the Stage I field work.

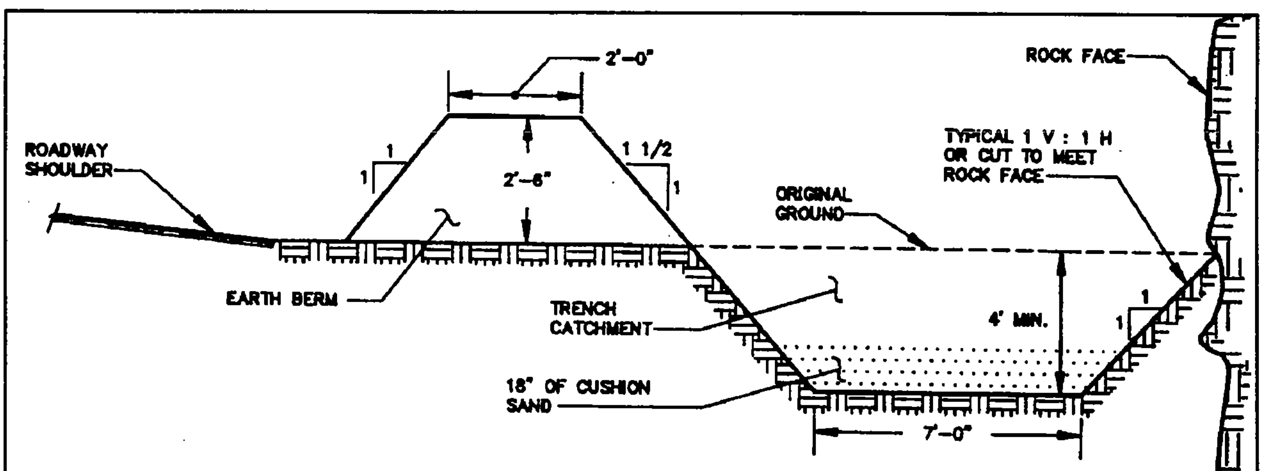


Figure 9: Catchment type B; typical section.

Table VI: Rock Mesh Schedule

WIRE MESH FOR ROCK SLOPES											
Location	MP	Start Station	End Station	Length	Avg. Length	Item 25203.1709(SF)	Rock Bolts Number of 1" DIA	Spacing	♦ (DEG)	D	Item 17203.171105(LG)
9	81.23NB	2+00	9+50	750'	78	58,500	16	50'CC	30	7'	112
11	86.80NB	1+00	6+50	550'	43	23,650	12	50'CC	30	5'	60
12	87.65SB	3+50	12+00	850'	35	29,750	18	50'CC	30	5'	90
13	89.31SB	1+50	3+75	225'	28	6,300	6	50'CC	30	5'	30
15	102.47NB	6+00	10+50	450'	30	13,500	10	50'CC	30	5'	50
15	102.59NB	12+50	16+25	375'	31	11,625	9	50'CC	30	5'	45
16	102.72SB	10+75	16+75	600'	36	21,600	13	50'CC	30	5'	65

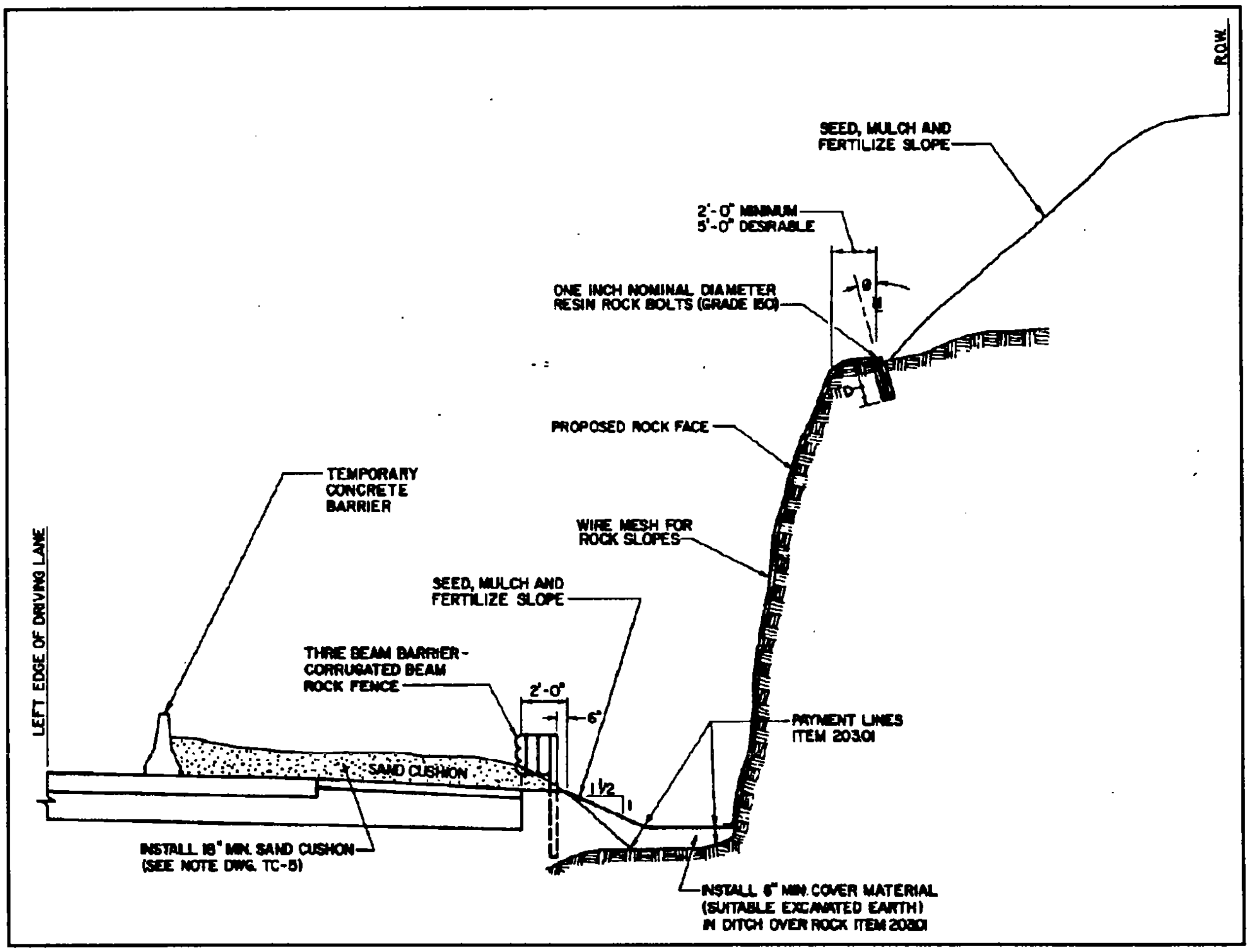


Figure 10: Rock mesh; typical detail.

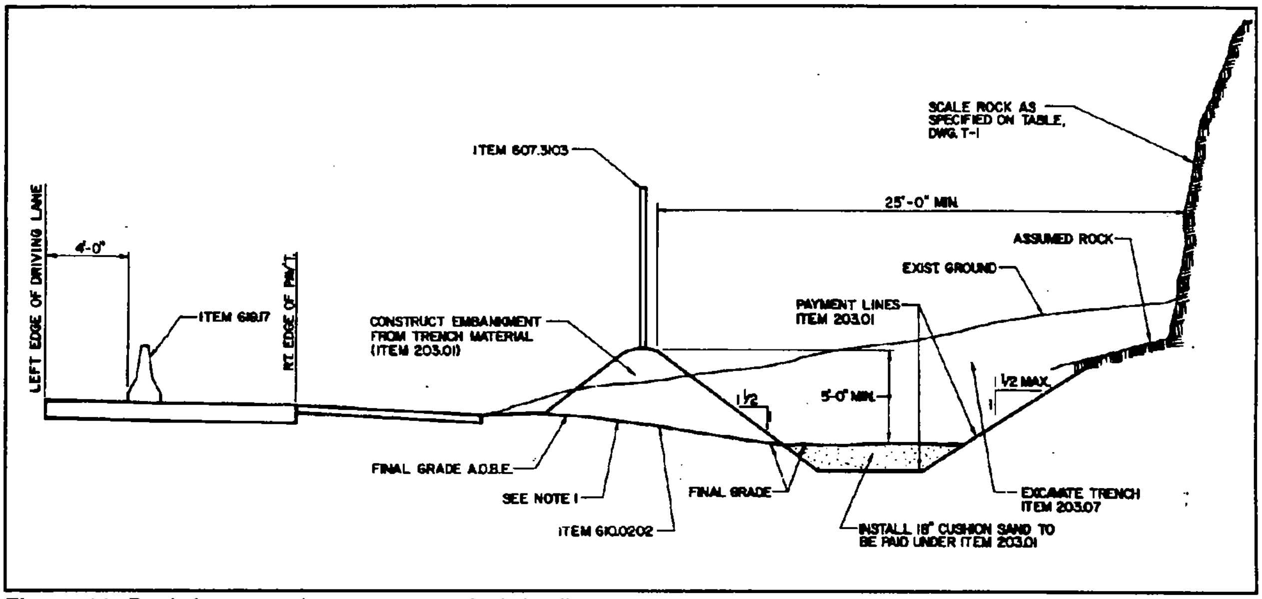


Figure 11: Rock fence catchment area; typical detail.

Acknowledgments

A number of people were directly involved in the performance of the work and with their assistance and input led to the fast track and successful conclusion to this project. The authors would like to give special thanks to: Peter Sutherland, former CHA employee, who was responsible for the formulation of the field investigation program and assisted in design remediation; James Quinn and Richard Cross, NYSTA, who assisted every step of the way during the field program; Raymond Rumanowski, CHA, who directed the construction sequencing and maintenance and protection of traffic portions; and Clay Bolton, NYSDOT Soil Mechanics Bureau, who provided information regarding current NYSDOT Specifications and remediation measures.

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DESIGN FOR A REUSABLE TEMPORARY ROCK CATCHMENT BARRIER

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New York State Thruway Authority

ABSTRACT

A number of recent projects along the New York State Thruway have required that traffic be maintained adjacent to work zones where rock was being excavated. The geometry of the slopes along with an excavation sequence that requires multiple lifts has produced situations where material from intermediate benches has to be dropped from substantial heights into relatively narrow catchment areas. A high level of risk of rock debris bouncing or splattering into active traffic lanes exists under these conditions. In most of the areas high traffic volumes precluded the intermittent traffic shutdowns that would normally be used. To resolve this problem a temporary, and reusable rock catchment system has been developed. This system has been designed to provide a level of security that will allow the full range of activities on an excavation site, with the exception of blasting, to be conducted virtually unrestricted directly adjacent to traffic.

Four major features of the system design are:

1. It is strong. The system is designed to withstand impacts of 8 foot-tons at the weakest point without sustaining critical damage.

2. It is reuseable. The major elements of the system are designed either to be self protecting or to be protected by a sacrificial element. Additionally, the modular nature of the design tends to limit the extent of damage in the event of a severe overload as well as make it easily repairable.

3. It is compact. By incorporating the standard work zone protection barrier into the system the space requirements have been limited to 5 feet. This includes an allowance of 4 feet for net deflection on impact.

4. It is readily modified. Should additional strength be required the system is easily modified to increase sliding and overturning resistance. Changes in height can also be accomplished simply.

This system has been used successfully on three projects to date. Initial results indicate actual performance greatly exceeds the design expectations.

INTRODUCTION

The New York State Thruway is a 559 mile toll road that makes up the backbone of the Interstate highway system in the state. Extending north from New York City to Albany and then west to Buffalo and the Pennsylvania state line the highway is sometimes referred to as the "Main Street of New York". Constructed during the early 1950s the Thruway was built to what were then state of the art standards. The rock slopes were designed with a slope of three vertical on one horizontal. Construction, however, predated the use of presplitting as a blasting technology, and the actual slopes varied from vertical to approximately one on one. With few exceptions the slopes were constructed with a narrow, by current interstate highway standards, setback, averaging approximately 17 feet from the edge of the driving lane to the toe of slope.

In the spring of 1988 the New York State Thruway Authority undertook development of a rock slope management system. As part of this effort all rock slopes along the superhighway were inspected and evaluated to determine the need for remedial work. As an outcome of this study 35 slopes were identified as needing immediate remedial work. The height of the slopes included in the program ranged from 30 to over 100 feet. Following the initial evaluation process a second evaluation of the slopes was conducted to determine the scope and urgency of the remedial efforts required at each location. The result of this process was a two-phase rock remediation program. The first phase consisted of those rock slopes where scaling or trimming were considered appropriate and cost effective or where protective measures such as draped wire mesh or catchment fences would prove adequate. Since the design of remedial work for these areas was relatively simple they were advanced as rapidly as possible so as to be let and completed that construction season. The second phase included those slopes where a more permanent solution was necessary or more cost effective. Work at these sites is ongoing.

The geometry of the work sites combined with high traffic volumes encountered at some of the locations, produced situations where it became very difficult to assure the safe maintenance of traffic during the work. Because the Thruway is a toll road it has been a long-standing policy of Thruway management to assure our patrons the safest and least restricted flow of traffic possible. This policy precluded intermittent traffic stoppages that would normally have been used. As an ameliorative measure, detour lanes were constructed at several of the sites to allow a permanent work zone to be established and still maintain the ability to provide a full flow of traffic for weekend and holiday periods. Even with the extra space provided by these lanes, work zones were quite narrow, and the risk of rock debris bouncing or splattering into the active traffic lanes remained substantial. During the first year of the remediation program the responsibility for protecting traffic from this hazard was included as part of the contractors obligation to maintain a safe work zone. It soon became evident, however, that the level of protection necessary required an effort that was not reasonable to expect under the standard "maintenance and protection of traffic" item. With these considerations in mind it was decided to provide a rock catchment system as part of further contracts.

DEVELOPMENT

The Rock catchment barrier was developed to meet the need for a temporary rock-fall protection device for construction sites that would permit the full range of construction activities on a rock excavation project (with the exception of the actual blasts) to be conducted virtually unrestricted directly adjacent to traffic. During the development process, four criteria were identified that the barrier system should meet.

The barrier system would need to be compact. With space in work zones already at a premium, little if any room could be spared. Since all sites would require work zone protection and pavement protection, it seemed prudent to integrate these into the design. By this method the extra space required for the system was limited to approximately three feet.

The system would have to be strong. At first the system was envisioned as being used primarily at scaling sites or where shot rock was being cleared from a temporary bench during a multi-lift excavation. After further consideration however it became apparent that the barrier would have to be installed prior to the start of work and remain in place throughout the duration of excavation. This meant that the barrier would be in place during blasting and therefore subjected to greater impact forces than had been originally thought.

Reusability was also very important. It was obvious from the start that the cost of providing the desired level of protection would be high enough to rule out an expendable system. Since it would be impossible and impractical to design a barrier system strong enough to prevent damage to the system, the design needed to include some sort of sacrificial element to limit the extent of damage in the event of an overload. This has been accomplished thru the use of a weak post design.

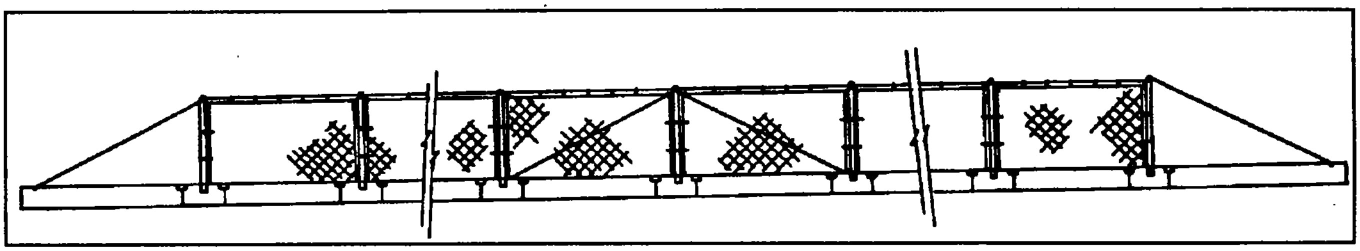

Figure 1: Front view of typical rock catchment barrier.

The posts have been designed to withstand a direct strike at the top of the post of eight foot-tons. This energy level represents a three hundred pound rock striking the post, at its most vulnerable spot, at forty miles per hour. Finally, any system would have to be flexible. Flexible not only in the sense that it could conform to any curvature in the roadway but also flexible in its ability to be modified to meet conditions at future work sites. Given the impossibility of determining what the loads on the barrier system would be, as well as how the various elements would actually perform, it was imperative to have backup plans of how to quickly" beef up" the system. To date, none of these have been needed.

The actual development of the system described here took place over a number of years as the result of problems the author had encountered on many stabilization projects. The work along the Thruway merely presented the opportunity to put the accumulated ideas of many years to use.

DESCRIPTION

The barrier system consists of two assemblies, the base assembly and a fence assembly. The base assembly is made up of three precast concrete units: a post base unit, a section of standard temporary concrete barrier and a connecting beam.

The post base units are Tee shaped with the top of the Tee at the back of the unit. The main body of the unit is approximately 5 feet square by 32 inches high, it weighs just over 5 tons. The post bases and barrier units are linked together with the same H key used to connect temporary traffic barriers.

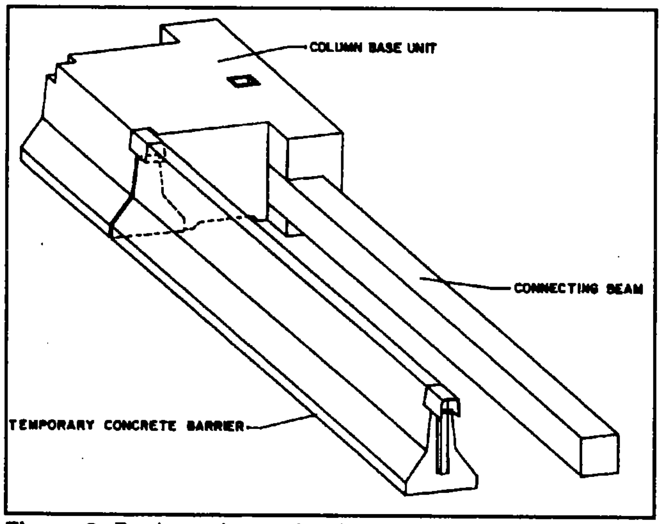

Figure 2: Rock catchment barrier base assembly.

Because the post bases have the same "safety shape" on the traffic side as the temporary barrier units and use the same connection; the completed assembly becomes a part of the work zone protection. The connecting beam fits in a notch in the Tee at the back of the post bases. The completed base assembly forms a hollow bin area, which is filled with earth material to a level one foot above the top of the beam. In addition to increasing the sliding and overturning resistance of the system, the backfill material also helps protect the base assembly and underlying roadway from rock impacts. The combined weight of the units in the base assembly, approximately 15 tons, along with additional weight of the earth backfill, allows the assembly to provide a high degree of resistance to sliding and overturning forces, yet the individual units are still small enough to be able to be handled with a moderate sized loader or backhoe.

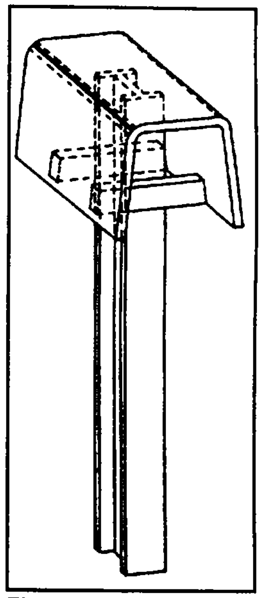

Figure 3: Typical H-key connector.

The fence assembly has four components: fence posts, wire rope mesh panels, a support cable, and chain link fencing. The 5WF16 fence posts slide into structural steel tubes cast into the post base units. The tubes are located four feet back from the traffic face of the barrier. This position has two major benefits. First, it greatly enhances the overturning resistance of the system. Second, it allows full deployment of the cable mesh panels without going beyond the face of the barrier. The cable mesh panels are woven in an 8 by 8 inch diamond pattern using a 5/8ths inch diameter cable for the border rope and 5/16ths inch diameter cable for the mesh. Fence panels are secured to the posts with 3/4 inch diameter shackles placed around the border rope and attached to the posts using holes in the outer flanges of the posts. The support cable is used prevent sagging of the cable mesh panels, which are attached to it with small shackles. Once these panels are attached they are overlaid with chain-link fencing to retain the smaller rock particles.

APPLICATION

As noted previously the system was designed initially as a barrier to prevent bouncing and splattering rock, from a scaling operation or the excavation of blasted material from an elevated bench, from reaching active traffic lanes. The system was expected to provide this level of protection without sustaining serious damage, thus requiring little or no maintenance. Additionally, it was expected to enhance the catchment area's ability to contain blast generated debris by both increasing its capacity and providing the ability to retain a moderate overload. For this purpose the catchment area is considered full when the angle of the debris slope is at the angle of repose and the toe of the slope reaches the top of the base units at the net line. On the projects where the barrier has been used the catchment area width available behind the face of the barriers has varied from 24 feet to 37 feet with slope heights ranging from 35 to 85 feet. When the system is to be used in these situations it is anticipated that some maintenance will be required with the amount dependant on how the blasting is conducted. To date the catchment barrier system has been used on three contracts and a fourth is currently starting. During the course of these projects two areas of concern have arisen, both related to blasting induced overloads.

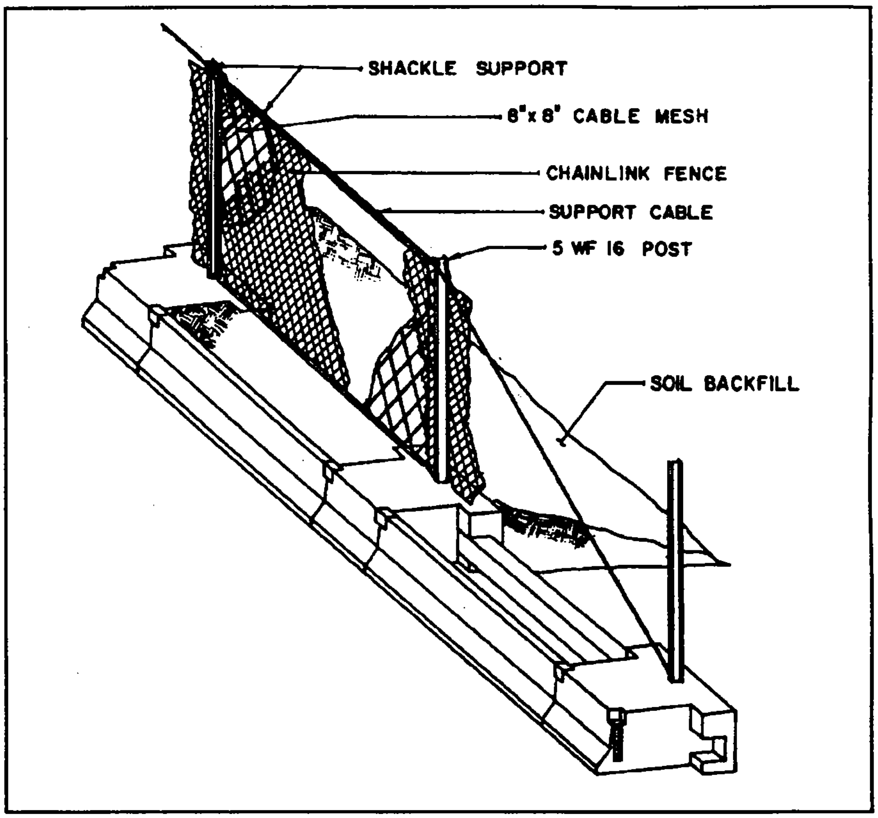

Figure 4: Rock catchment barrier fence assembly.

The most common problem experienced has been post failure. In most cases the cause of the failure has been that the volume of material blasted exceeded the capacity of the catchment area. On projects where this has been a consistent practice post failures have run in the 50% range. On one project 14 of the original 29 posts required replacement. While statistically this is a high failure rate, it is not considered to be a major problem for several reasons. First, the number of post failures can be greatly reduced if the contractor modifies the blasting operation so that the volume of material blasted nearly matches the catchment area's capacity. This can

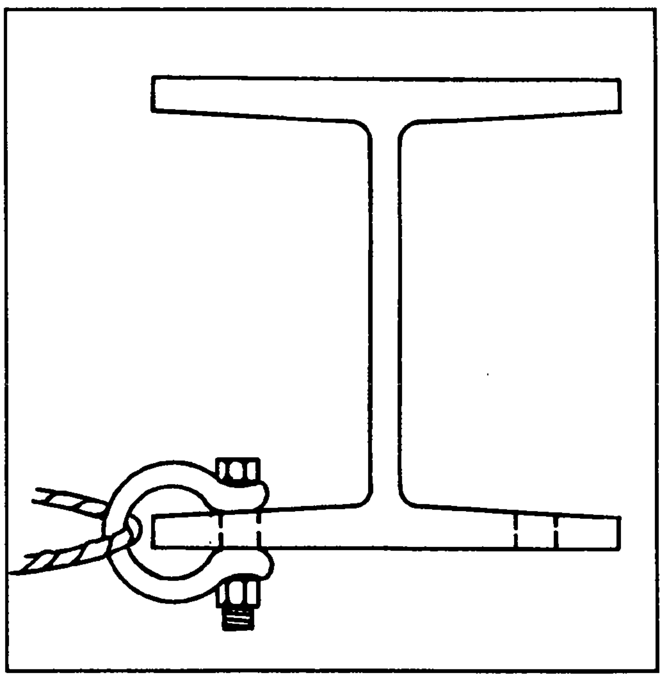

Figure 5: Typical shackle connection.

easily be accomplished by limiting the depth of the lift blasted. Second, it was anticipated that blasting procedures would be a source of problems, since this is primarily the responsibility of the contractor the responsibility for maintaining the barrier was given to the contractor. Third, post failure does not mean system failure. Even during blasts where the system has been overloaded to the point where 2 or more posts (6 in one case) have broken they have continued to link the nets and maintained the integrity of the system. In all overload situations experienced to date the system has retained within the catchment area 80% or more of the material impacting the nets. In some instances the depth of material retained was over 4 feet above the top of the base units. Finally, The current 5WF16 post design was chosen to provide a simple and easily obtainable post. The theory was to make the posts cheap and easy to replace. While this has worked, field experience indicates that using a stiffer post with a breakaway base would provide better service and ultimately a lower cost.

The second problem involves localized movement of the entire barrier system. On two occasions the force of debris impacting the catchment areas was sufficient to move the barrier system. The amount of movement was great enough to require excavation of the backfill material in three bays in order to return the system to its original location and replace broken sections of temporary barrier. Little or no damage was sustained by the posts or nets from these events. These movements were either related to the way blasting operations were being conducted or placing the barrier system too close to the slope face. No modifications to the barrier system seemed warranted.

SUMMARY

On the Thruway projects on which it has been used, the temporary rock catchment barrier system has been proven to be a very satisfactory means of protecting traffic from falling and bouncing debris. Even though it was not specifically designed as a containment device for use during blasting, it has been very effective for this purpose as well. Modifications to the posts currently under consideration should make it even more effective. While no system can provide absolute security, the level of protection provided by the system has shown it to be a valuable tool for use on remediation projects.

THE EVOLUTION OF ROCK EXCAVATION AND STABILIZATION IN NEW YORK STATE EMPHASIS ON THE WEST POINT QUADRANGLE

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INTRODUCTION

The invention of dynamite by Alfred Nobel in 1866 made possible rapid excavation of rock. Dynamite has directly influenced the feasibility and economics of mine excavation, quarry products, transportation corridors (canals, highways and railroads) and water supply.

Rock excavation was a major requirement for constructing the transportation system in the West Point area. The work required established the historic foundation for rock excavation (Circa 1909) and rock slope stabilization (Circa 1970) in New York State. Six tunnels (five railroad and one highway); four major highways (Routes 9D, 9W, 218 and 6-Bear Mt. Bridge Highway); one major aqueduct (Catskill) that supplies New York City Metropolitan area and one nuclear power station (Indian Point) have been constructed in the high rising rock promontories along the east and west shores of the Hudson River in the West Point Quadrangle.

Each of these unique projects has utilized innovative rock excavation procedures to establish major transit links and provide water and electricity to New York City.

WEST POINT

West Point was established in 1778 as a military post during the Revolutionary War. The post was well positioned. It commanded a narrow land corridor from the south and a relatively narrow section of the Hudson River on the east. The site was protected further by the steeply rising promontory to the north and west.

The West Point post became the United States Military Academy by an Act of Congress in 1902. The academy trains young people for appointment as officers in the U.S. Army.

CATSKILL AQUEDUCT

Eight major reservoirs comprise the water supply system for New York City and the surrounding metropolitan area. The Catskill Aqueduct was the first aqueduct. It connected the Ashokan Reservoir (some 100 miles North of New York City) and the Schoharie Reservoir to the Croton Reservoir in (Southern Putnam and Central Westchester Counties. These two reservoirs were connected by an inverted siphon under the Hudson River at the Cornwall - Breakneck Crossing before reaching Jerome Park Reservoir in Fordham, The Bronx. The construction of this major project took place in the period from 1900 to 1913.

The Rondout Reservoir was then built and connected to the Croton Reservoir by the Delaware Aqueduct Crossing at Chelsea in Southwestern Dutchess County. Subsequently, the Pepacton and Cannonsville Reservoirs were constructed and connected to the Rondout Reservoir by extending the Delaware Aqueduct some fifty miles to the west.

The system of reservoirs located as far as 125 miles northwest of the Jerome Park outlet comprise one of the largest water supply systems in the world.

The most difficult phase of the project was the construction of the inverted siphon that took the aqueduct tunnel more than seven hundred feet below the Hudson River. Exploratory drilling operations required three years (April, 1906 to July, 1909) to establish the most favorable crossing site. The Cornwall-Breakneck Crossing Site was selected because of the quality of the bedrock. Construction work of the crossing began in 1910. Eight shafts were driven to enable removal of the blasted rock from the tunnel headings. Work progressed in both directions from each shaft. The shaft depths varied from 342 to 816 feet. Muck (blasted rock) was loaded by hand on mine tipple cars and hauled from the tunnel to the shafts by mules, where it was removed by hoists. The mules were later replaced by electric trains.

The drilling, blasting and surveying were so precise that break-through crews coming at each other from opposite directions found themselves within fractions of inches of each other.

One compressor, located several miles to the west, provided all the air to power the hoists and to ventilate the tunnel. The compressor was located adjacent to the O and W Railroad, which delivered coal to the boilers that powered the compressor.

The siphon section of the Catskill Aqueduct was completed in 1913 at a cost of three million ($3M) dollars. The entire aqueduct is gravity fed; therefore, the siphon section had to be airtight for the system to function. High grade concrete was used to line the siphon section to insure the airtight condition.

The three million dollar project was a phenomenal operation for the period and yet most area residents were unaware that this work was being performed. Very few local people were employed on the project and the New York City Board of Water Supply Police kept the notoriously unruly workmen away from local establishments.

ROUTE 218 - OLD STORM KING HIGHWAY

Chronologically, the second rock excavation feat of significance accomplished in the West Point area was the construction of a highway between "The Point" and Cornwall (three miles to the north) along the west side of the Hudson River.

The desire of New York City residents to vacation in the Catskill Mountains the north of West Point, occurred simultaneously with the rapid progression of automobile development.

As a result, demand for a more direct route to the mountains became a State political issue. The "trail end" location of West Point and the Army's need for a civilian work force to staff the post coincided with a substantial roster of unemployed personnel in the Cornwall-Newburgh area.

Political forces promoted plans for the construction of the roadway and in 1903, the first survey for the project was started. The survey was no easy matter. The centerline of the proposed highway was plotted on the steep, treacherous terrain by men lowered several hundred feet on ropes. Totally inaccessible areas were marked by canisters of paint attached to rockets fired to the rock surface from boats on the river below.

The plans for constructing the highway were completed in 1907. A low bid of $187,000 was received and the Contract awarded. Work never began, however, since a group of high financiers who represented themselves as a mining company purchased the land on the proposed highway "right of way" and demanded an outlandish sum of money for the property.

To circumvent the problem, a new survey was taken and the grade limited seven percent rather than the original five percent. These changes raised the elevation of the highway 400 feet above the river, obviating need for constructing a tunnel. Bids were opened in 1913. The low bid received was $197,000 for 1.27 miles, less than half of the length of the total highway.

The designers returned to work and devised a wedge shaped cutout of the side of the mountain. The deepest cut was 130 feet above the west ditch line. On July 15, 1915, a low bid of $273,736.70 was received. Thus began the construction of the highway for $100,000 more than the initial bid and a ten year delay.

The finished roadway was 24 feet wide with a macadam pavement. The highest section of retaining wall along the east side of the highway is 38 feet and the greatest depth of fill is 75 feet. The total excavation quantity was 119,000 cubic yards. Most of the excavation was drilled and blasted rock. The rock was used to construct the retaining wall. Sixty percent of the wall was laid dry and very little maintenance has been required in the 75 years of its existence.

The terrain is so precipitous that equipment had to be hauled to the mountain in pieces and assembled at the site. The placement of a five ton crane alone cost $4,500. The crane parts had to be hauled seven miles and hoisted 1,300 feet over the top of the mountain. Workmen scaffolds were suspended more than 200 feet to allow drilling the bedrock to depths of ten feet to blast the rock to form a shelf. The shelf was then used to drill ten feet to the next bench and so on until the highway grade was achieved. Benches were constructed in sections, 100 feet in length.

The road was completed in 1918 despite the war with its embargoes and reduction in supplies, men, equipment parts and explosives.

Rock falls have hampered traffic but the highway continues to be functional. Maintenance work is ongoing to pin the rock in place to prevent its falling on the heavy volume of traffic. Further traffic safety is provided by road closure following a one inch rainfall in a 24 hour period.

A large volume of rock slid from the rock slope on October 23, 1982 destroying 125 linear feet of the stone masonry parapet along the east side of the roadway. The highway was closed upon advisement of a NYSDOT engineering geologist until such time as a remedial rock slope stabilization contract could be let.

The following remedial treatment was recommended before the road could be reopened.

Rock Slope Scaling - 1,000 cubic yards
Resin Rock Bolts - 2000 linear feet
Stone Parapet Replacement - 145 linear feet
Concrete Support Buttress - 7 cubic yards

A 70 ton crane with a 120 foot boom was used to provide access to the slope for scaling with a clam bucket and a headache ball. A man cage was utilized to allow drilling for placing explosives to remove fixed pieces of rock that would ultimately become dislodged by frost wedging. High winds required the use of guy lines on the man cage for workmen to guide the equipment into place. Final scaling was performed by the crane dragging a blasting mat over the rock slope face. High pressure water works very effectively for scaling slopes in crystalline rock.

From the quantity of remedial rock slope stabilization work required to maintain a safe highway, it must be inferred that rock ages as do our pavements and bridges. When this Route 218 roadway was closed for maintenance work, the pavement in front of the rock slope became littered with rock fragments of all sizes. My proposed explanation for this exfoliation process is two-fold. First, frost wedging (moisture penetrating fine cracks in the rock expands upon freezing) breaks the rock along the slope, dislodging it and allowing it to fall. Secondly, the rock continues to expand to neutralize the stress placed on the rock by previous geotectonic forces (including the two mile thick pleistocene glacier). The released stress produces rock cracking and the frost wedging cycle continues.

THE BEAR MOUNTAIN - HUDSON RIVER BRIDGE

The Bear Mountain - Hudson River Bridge was dedicated and opened to the public on November 26, 1924. Thus was completed, one of the longest span bridges in the world devoted exclusively to highway traffic. The bridge provided a greatly needed route for east-west traffic between New Jersey, Pennsylvania and upstate New York to the New England states. It also shortened the route within New York State itself by connecting the shores of the Hudson River.

Crossing the Hudson River from Anthony's Nose to the historic Fort Clinton at a location approximately 42 miles north of New York City, the bridge provided the first highway crossing of the Hudson River south of Albany.

The bridge and the Bear Mountain Highway to Peekskill were built at the same time, thereby, completing the east-west corridor which linked highway systems on both sides of the river. The bridge and highway were built with private monies of Edward N. Harriman, owner of the Southern Pacific Railroad. Edward's son, W. Averell Harriman was Governor of New York State (1950-1958) and later, U.S. Ambassador to the U.S.S.R.

Since the bridge and highway were privately owned, tolls were collected ($1.00 per vehicle and 10¢ per passenger or pedestrian). Today, round trip passage costs 75¢ per vehicle. This is one rare case where a cost has been reduced for a product or a service. However, the tolls received compensated Harriman for the approximately $4 million laid out by the Bear Mountain Bridge Corporation. Eventually, the townships of Cortlandt in Westchester County and Haverstraw in Rockland County began the pursuit of the value judgement of the property taxes. The Bear Mountain Bridge Corporation refused payment for several years until taken to court by both townships. The dilemma was ultimately resolved in 1940 when New York State purchased the bridge for nearly $5 million and thus, the New York State Bridge Authority was born.

The State of New York also purchased the Bear Mountain Highway and became responsible for what has been determined by the NYSDOT Engineering Geology Section as the highway with the most hazardous rock slopes in the State. A rock slope assessment "risk factor" developed by Golder Associates for the state of Oregon in 1987 and utilized, with some adjustment, by New York State in 1989 established this roadway as the most hazardous. With a maximum of 81 points for ten risk categories, the maximum risk point total on the Golder scale is 810. New York State uses an eleventh category, "Backslope Above Cut" resulting in a maximum risk point total of is 891 points. Three sections of the Bear Mountain Highway have been rated at 891 points. The presence of the Metro North Railroad, at the base of the mountain, 150 feet in elevation below the west side of the highway, was the overwhelming factor that made the Bear Mountain Highway the highest risk roadway in New York State.

A contract completed in 1990 which dealt primarily with rock slope set back and stabilization together with a deep rock catchment ditch (Ritchie Ditch Criteria) cost $2 million of the $3 million contract. Eliminating only one of the aforementioned three high risk sections of the highway required 50,000 cubic yards of rock excavation, 250 linear feet of resin rock bolts and 1,200 linear feet of rock catch fence installation.

The highway had to be closed for six months to perform the work. Additional work is required to stabilize the rock slopes to improve highway safety. A better detour must be constructed prior to the implementation of further contract work. The magnitude of private and commercial vehicular traffic caused a significant increase in accidents on the sinuous Route 403 (Garrison Road) to the north in Putnam County.

Pressure to open the highway was exerted by the N.Y.S. Bridge Authority since Bear Mountain Bridge tolls were reduced by 50 percent during highway closure. Metro North Railroad was apprehensive during blasting operations due to the contractor's inability to retain all blasted rock within the highway right of way. Numerous large pieces of blasted rock escaped the highway easement and cascaded to the railroad at the toe of slope. The time "window" provided by the railroad was sufficient to enable rock removal without disruption of rail traffic. Metro North was very cooperative throughout the duration of the contract.

As on Route 218 (Old Storm King Highway), rock slope maintenance must be conducted periodically on Bear Mountain Highway to reduce the risk of falling rock striking a vehicles or conversely vehicles striking a rock in the roadway. Both highways have very narrow (<4 - feet wide) shoulders which limits vehicular maneuverability around obstacles. A potential solution to rock falls is to move the slopes back and construct ditches at the toe of the rock slopes which comply with the "Ritchie Ditch Criteria". Compliance with this criteria for the entire Bear Mountain Highway would cost at least $50 million at 1992 prices.

An earlier 1972 rock slope stabilization contract was let as a cost-plus contract with 90 percent Federal aid following Hurricane Agnes. Approximately 1,000 linear feet of rock bolts were installed on the Bear Mountain Highway at that time. Three-quarter-inch diameter bolts with serrated expansion shell anchors (then the state of the art) were used on this project.

Poor quality chain link rock catch fences were installed and though properly located, they were not high enough to catch rock which bounded down the slope from high above. Work on this project was prematurely stopped due to depletion of emergency funds.

ROUTE 9W (NEW STORM KING HIGHWAY)

Work began on January 23, 1937, with rough grading for the new Storm King Highway under a New York State Department of Public Works Contract. The total four lane 5.28 mile highway from West Point to Cornwall was completed and opened to traffic on September 26, 1940. The entire cost of the project was $1.681 million.

Coincidentally, the Bear Mountain Bridge came under the jurisdiction of New York State on the same date (9/26/40) as the Storm King Highway opening. (The relevance of this coincidence has escaped the author.)

In 1982 following accidents resulting from falling rock striking automobiles on the new Storm King Highway, a rock slope stabilization contract was let for the two miles of 9W north of West Point for $237,000. The estimate of work included 600 linear feet of 1¼ inch diameter resin anchor rock bolts at $30 per linear foot. Prestressed resin rock bolts had at that time become the "State of the Art" rock bolt and this was the first project in New York State on which this rock bolt type was installed. The contract also included 2,075 cubic yards of rock slope scaling (Removal and Disposal of Hazardous Rock with Blasting) at $95 per cubic yard.

All excavation items in the NYSDOT Specifications are paid for by the cubic yard. Since in place measurement of scaled material is virtually impossible, all scaled material is weighed, and a conversion factor based on the specific gravity of the rock is provided in the Contract Proposal. The granite gneiss bedrock within the limits of this contract provided a conversion factor of .00021 cubic yards per pound (one cubic yard of this rock weighed 2.381 tons).

Once the work had begun, it was determined that a substantial increase in both scaling and rock bolting would be required if the highway was to be rendered safe. A demonstration of the instability of the rock slopes of the highway readily convinced the regional construction engineer that the rock slopes should be stabilized to the satisfaction of the project engineering geologist.

The aforementioned agreement resulted in the removal of an additional 2,000 cubic yards of unstable rock and an increase of 500 linear feet of rock bolts. The additional work resulted in the doubling of the cost of the contract to $490,000. No major rock related accident has occurred since the stabilization of the rock slopes on this section of highway was completed.

A unique scaling technique was used on this project which to the author's knowledge was used only once before in New York State. It was previously used to scale the columnar basalt along Route 9W in Haverstraw in Rockland County.

The method employed a single acting pile driver with an attached bull point. The point was a steel pin similar in size to that found on a pneumatic hoe ram. The pile driver with bull point was suspended from a crane. The pin was directed to a location behind a seemingly unstable block of rock on the slope by a coordinated effort of the crane operator and a laborer assigned to a guy line and an electrically operated switch to activate the pile driver.

The crane operator lowered the pile driver as the pin began to penetrate the joint behind the unstable rock mass. The unstable rock came down quickly. This piece of equipment was very effective and it undoubtedly contributed to the overrun in the scaling item. Often, an attempt to remove one definitely unstable piece of rock would cause an avalanche of unstable material to cascade to the roadway.

A new contract for presplitting badly deteriorated rock slopes to the north of the aforementioned project has been recently let. Unfortunately, work will not be underway at the time of the scheduled May 30, 1991 field trip. Failure of the New York Legislature to pass a budget has mandated a delay in award of all new contracts.

CONCLUSION

Only the major rock excavation projects have been developed in this report. The projects have been portrayed to illustrate the development of the "State of the Art" changes in rock excavation, rock stabilization and most importantly employee safety.

Rock excavation began on the Catskill Aqueduct with jack leg drills to make the holes for unstable high percent nitroglycerine dynamite. Many men lost their lives in the construction of this aqueduct. Improved, safe, specialized explosive products combined with carbide drill bits and pneumatic air track drills have produced clean stable rock slopes (New Storm King Highway, Stoney Lonesome Road Interchange, Metro North Railroad Tunnels).

The improved "State of the Art" rock bolt has progressed from the original wedge anchor bolt (Stoney Lonesome Road Interchange with Route 9W) to the serrated expansion shell anchor bolt (New Storm King Highway) to the prestressed resin anchor bolt (Bear Mountain Bridge Highway).

Workman safety has greatly improved from suspending drillers 200 feet down a shear rock face (Old Storm King Highway) to working in roofed man cages raised by a crane as in recent stabilization projects on Old Storm King Highway and Stony Lonesome Road Interchange.

Presplitting began with the construction of Stony Lonesome Road Interchange. One of the deepest presplit rock slopes (118 feet) was presplit above a 30 foot high bench in 1970. Presplitting has enabled the construction of a smooth, stable rock slope which requires minimum maintenance.

The rock slopes constructed along the Old Storm King Highway, Bear Mountain Bridge Highway and New Storm King Highway using production explosives exclusively, are unstable and demand continual maintenance to keep the roadways clear of rock debris.

Ultimately, all of the rock slopes will be rehabilitated as "state of the art" methods for construction and stabilization are employed. This cannot be performed without capitol expenditures which are always difficult to obtain. A regular systematic method of stabilization is recommended in the interest of safety of the traveling public.

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GEOTECHNICAL EXPLORATION OF COMPLEX TUNNEL SITES

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ABSTRACT

Five recent highway rock tunnel projects had received limited geotechnical exploration previous to design. Additionally, these projects were at remote, mountainous sites with limited access and high environmental sensitivity. These complexities posed significant challenges to the geologists and geotechnical engineers during the geological exploration phase of the projects. These five projects consist of: (1) repair of a rock slope failure which destroyed a portal and connecting tunnel along Interstate 40 in North Carolina about 5 miles east of the Tennessee border; (2) the Cumberland Gap Tunnel Project along U.S. Highway 25e in Cumberland Gap National Park in Kentucky and Tennessee; (3) the five two-lane tunnels being constructed in Glenwood Canyon, Colorado along Interstate 70; (4) the Interstate Route H-3 project in Hawaii which will pass through twin two-lane tunnels each about 1 mile long through volcanic basalt flows; and finally (5) a project in Utah, the most recent, where the two-lane U.S. Highway 189 through Provo Canyon is being widened to four lanes. Unconventional exploration techniques helped to overcome exploration complexities at these sites.

INTRODUCTION

Geotechnical exploration is the primary work that highway geologists commonly undertake. The complexities of geologic environments alone pose significant challenges to the successful geologic investigation, data interpretation, design, and construction of highway projects. When the sites are remote, mountainous, and environmentally sensitive, this only complicates the process. So it was on I-40 in North Carolina, U.S. 25e in Kentucky and Tennessee, I-70 in Colorado, H-3 in Hawaii, and U.S. 189 in Utah. Each of these sites required careful and accurate geologic exploration for design and construction. Each of these projects required exploration methods above and beyond conventional drilling, sampling, logging, testing, and interpretation methods. These projects and the geologic exploration methods used are summarized in Table 1 and are the subject of this paper.

INTERSTATE 40 STERLING MOUNTAIN TUNNEL FAILURE

On March 5, 1985, a massive rock slide involving roughly 14,000 cubic yards of rock debris occurred at the eastern portal area of the twin tunnels which carry Interstate 40 through Sterling Mountain in North Carolina about 5 miles east of the Tennessee border. The 150-foot canopy or tunnel extension which had been built at the eastern end of the westbound tunnel to protect the interstate from falling rock was destroyed and the eastbound lane was also completely blocked.

Immediately after the failure, while debris was still being cleared away, geologic exploration commenced with a search of existing data and initial reconnaissance. The primary goals of the hastily-planned geotechnical investigation program were to determine a plausible cause of the failure and to develop parameters for rock reinforcement design. A similar event was to have a low probability of occurrence in the future. Furthermore, this information had to be collected in the minimum amount of time.

The exploration program that was developed was aimed at describing the orientation, frequency, and characteristics of rock mass discontinuities and locating the weak, weathered, phyllite interbed that probably was the base plane of the failure. The former was accomplished by measuring joints on the exposed rock face adjacent to the tunnel portal. The slope was about 300 feet high and mapping was done by geologist/rappelers from the North Carolina Department of Transportation Geology Section. Because the phyllite interbed dipped into the slope relatively high up above the road level, the latter aim of the program was accomplished by drilling boreholes from inside of the remaining tunnel. Because the bedding was prominent in the rock core, joint orientations could be roughly inferred relative to bedding based on the geologic mapping results without using oriented core drilling methods. The data gathered in these ways provided a relatively accurate picture of the rock mass properties from which the cause of failure and remediation methods could be developed.

Good to excellent quality Longarm Quartzite is above the tunnel under which is poor to fair quality Longarm Quartzite interbedded with siltstone, slate, and phyllite. The knob of rock that failed was bounded by a fault trending nearly east-west, a major joint set trending northeast and bedding which strikes northwest. Because of the pre-existing discontinuities in the rock mass, the abundant water available to accelerate weathering along these discontinuities, and probably freeze-thaw pressures, the block of rock finally failed destroying the westbound portal section of the tunnel.

The tunnel and portal were repaired on a fast-tracked schedule so that normal traffic flow could be restored prior to the onset of the next winter. Repair included demolition of the damaged tunnel and portal, scaling and blasting loose rock from the remaining slope, rock dowel and mesh reinforcement, reconstruction of the concrete tunnel and portal, and construction of a Reinforced Earth rockfall catchment area above the portal. More detail about this repair is given in "Analysis and Rehabilitation of Aging Rock Slopes" by Abramson and Daly (1986).

CUMBERLAND GAP TUNNEL PROJECT

Currently, U.S. Highway 25e between Cumberland Gap, Tennessee and Middlesboro, Kentucky carries a traffic load of more than 18,000 vehicles per day over a steep, winding, two and three lane route that passed over the Cumberland Gap in the Cumberland Gap National Park. The historical nature of the current route, coupled with the technical problems of roadway improvements in such difficult terrain, has led to plans for relocating the highway into twin two-lane, 4,100-foot-long, 40-foot-wide, tunnels through Cumberland Mountain.

At this site, colluvium overlies bedrock consisting of interbedded sandstone, limestone, and shale dipping at 35 to 50 degrees and striking nearly perpendicular to the tunnels (Abramson and Slakey, 1990). Exploration was aimed at characterizing the colluvium for the purpose of portal design and at characterizing the bedrock that the tunnels will go through.

Initial geologic exploration included literature review, field reconnaissance, aerial photo interpretation, seismic refraction, and borehole drilling using conventional rigs as well as portable rigs on the steep side slopes adjacent to the tunnel portals (Sullivan and Leary, 1987). Once the overall geology at the project site was defined and general geotechnical information was obtained, a series of near-horizontal borings were drilled from the Kentucky portal site along the tunnel alignment about 2,000 feet into the mountain. Difficulties during this phase of the work included colluvial slope failures that buried the rigs twice during drilling. In addition to conventional rock core logging information, resistivity, seismic, and electromagnetic data was obtained in the boreholes (Brumund et. al., 1985). Next, an exploratory tunnel was constructed to observe directly ground and support behavior due to tunneling in this rock mass.

The exploratory tunnel was constructed between December, 1985 and December, 1986. It was constructed in the crown of the future southbound tunnel. The rock mass was found to consist of massive to very blocky rock having a Q-rating typically between about 1 and 3. This places it in the "fair" tunneling category (Parsons Brinckerhoff, 1988). The exploratory tunnel also disclosed the presence of coal, methane gas, solution features and caverns, and groundwater under pressure.

The twin highway tunnels are currently under construction. Another interesting feature of this project is the use of soil nailing to stabilize the colluvial slopes in the portal areas. These walls were instrumented and wall performance is described by Nicholson (1986).

GLENWOOD CANYON TUNNEL PROJECT

In Glenwood Canyon, Colorado, 12 miles of U.S. Route 6 is being converted into a four-lane section of Interstate 70. The two tunnel projects consist of the westbound

Reverse Curve Tunnel, a 600-foot-long, two-lane bore through a nose of rock, and the Hanging Lake Tunnel which consists of twin 40-foot-diameter, 3,500-foot-long mined tunnels, and a 500-foot-long cut-and-cover tunnel, ventilation, and control facility. The Reverse Curve Tunnel was completed in spring 1989 and the Hanging Lake Tunnel construction began in early 1990. The project site is located about 140 miles west of Denver.

The geology in this portion of the canyon consists of a complex sequence of pre-Cambrian igneous intrusive and metamorphic rocks that underlie the bedded Sawatch quartzites and other sedimentary rock formations that form the prominent cliffs along Glenwood Canyon (Woodward-Clyde Consultants, 1988). Geologic exploration included surficial geologic mapping in 1978, exploratory boreholes and laboratory testing in 1981, and an exploratory tunnel along the eastbound tunnel alignment in 1985. Horizontal borings and Goodman Jack tests were performed from the exploratory tunnel to the westbound tunnel horizon.

The Reverse Curve Tunnel was driven through stratified, moderately jointed, blocky and seamy dolomite, siltstone, sandstone, and quartzite. No groundwater was encountered. Two distinguishing conditions relating to tunnel construction were the proximity to the canyon face and the presence of persistent joints with relatively large apertures. Also, the tunnel was constructed adjacent to and in very close proximity to a heavilly travelled in-service highway facility. The rock mass as classified using Barton's tunneling quality index or Q-value ranged between about 10 and 14 which places it in the "good" tunneling quality category.

The Hanging Lake Tunnels were constructed in massive to very blocky and seamy quartz diorite, migmatite, granite, and pegmatite. Little groundwater was encountered. Q-values ranged between about 1 and 10 for most of the tunnels. This places them in the "poor" to "fair" categories. However, about one quarter of the tunnel lengths were in "good" to "very good" rock with Q-values greater than 10. Further information about these tunnels is given in the paper by Abramson and Slakey (1990).

TRANS-KOOLAU TUNNEL PROJECT

The Interstate Route H-3 project in Hawaii is planning, design, and construction of a 10-mile-long final segment of new highway that traverses the Koolau mountain range on the island of Oahu (Figure 1). The project includes a 1-mile-long tunnel through the mountain range made up of weathered basalt. The twin-bore Trans-Koolau Tunnel will have two lanes in each direction and a roadway width of 38 feet including shoulders. Construction began on the project in late 1987 and is expected to be complete in late 1994.

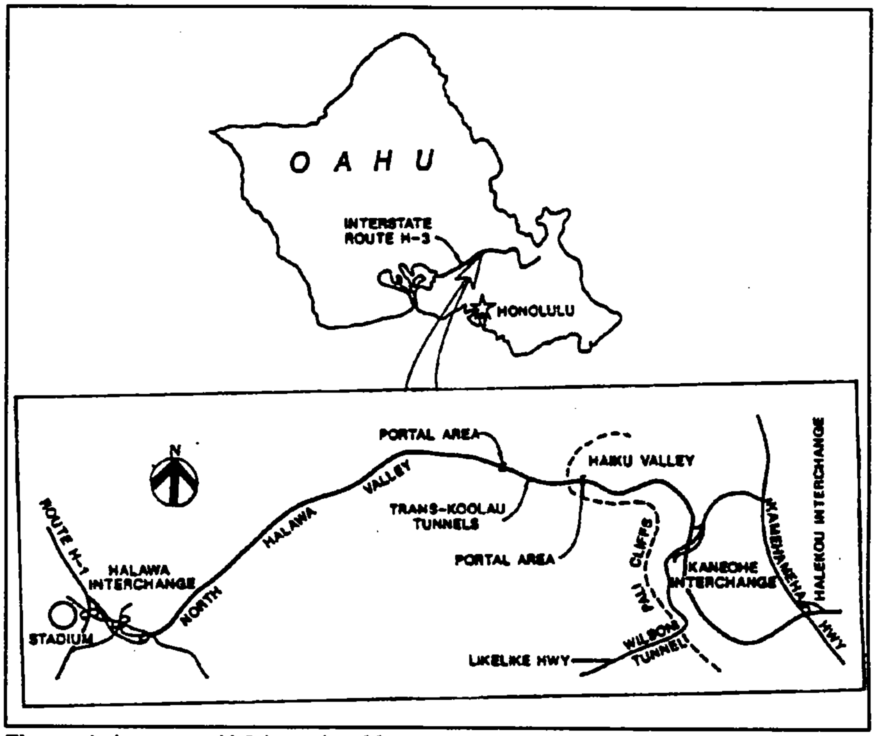

Figure 1: Interstate H-3 Location Map.

Geologic information for the project was needed early-on before there was any conventional access to the site. Thirteen vertical and horizontal borings were drilled with the use of helicopters for hauling the drill rigs, workers, and materials to the boring locations. Foam drilling techniques were used to conserve on drilling water which had to be flown to the drill sites in barrels. Special drilling and logging methods were developed for the

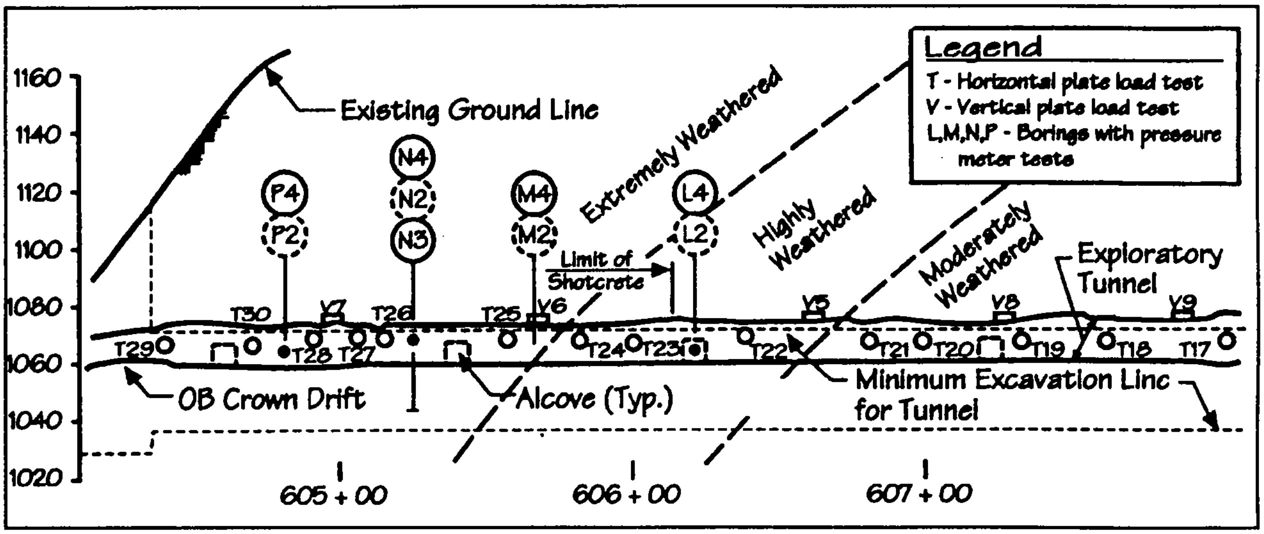

Figure 2: Geotechnical Testing Program - Interstate H-3 Exploratory Tunnel.

project to characterize the saprolite and basalt (Abramson and Hansmire, 1988). The final stages of the exploration program consisted of an exploratory tunnel and in situ geotechnical testing program (Figure 2) which included pressurementer and plate load tests, laboratory tests, and tests on alternative rock support methods (Boyce and Abramson, 1991).

The main exploratory tunnel was located in the pillar between the two future main tunnels to act as a drainage gallery in the event that large quantities of trapped dike water were encountered. There were also crown drifts (in the crowns of the future main tunnels) where the tunnels were expected to encounter deep zones of weathering. The exploration indicated that the extent of tunneling in saprolite (extremely to highly weathered basalt) was limited to a few hundred feet in each tunnel near the North Halawa Valley (westernmost) portals.

Most of the rock mass consists of moderately blocky and seamy, slightly to moderately weathered basalt "pahoehoe", "aa", and "clinker" common to the Hawaiian volcanic islands. Jointing tends to be closely spaced and discontinuous. The flows generally dip about 5 to 10 degrees and strike nearly perpendicular to the tunnel axes (Parsons Brinckerhoff- Hirota Associates, 1990). No high water inflows have been encountered from trapped dike water during tunnel construction. However, perched water zones are present above thin red clay layers formed from extremely weathered ash zones. Q-values range between about 1 and 5 placing the rock mass in the "poor" to "fair" tunneling quality categories.

Main tunnel construction has been divided into two contracts. The Honolulu-bound tunnels have presently been excavated to a distance of about 1,000 feet from the portals. Construction of the Kaneohe-bound tunnels is just beginning.

PROVO CANYON TUNNEL PROJECT

UDOT's Provo Canyon Tunnel Project consists of a two-mile-long segment of U.S. 189 in a section between Wildwood and Vivian Park, Utah called the "Narrows". The existing two-lane facility is being expanded to four lanes along the Provo River. The terrain consists of a series of alternating rock ridges and talus-filled valleys. Most of Provo Canyon is wide enough to accommodate four lanes with shoulders and median without crowding the adjacent river or creating excessively high rock cuts, fills, and/or retaining walls in this environmentally sensitive area. At the project site however, rock tunnels and talus slope cuts are required for the expansion through the Narrows.

The project is located in the Wasatch Mountain Range. The terrain in this area is generally rugged, consisting of steep, stream cut valleys, which terminate in relatively narrow ridge crests. The alignment is located along the north side of Provo Canyon, which is generally oriented northeast to southwest, cutting the Wasatch Mountain range. The Provo River occupies the bottom of Provo Canyon, flowing to the southwest.

The western front of the Wasatch Mountains forms a steep west facing slope formed by repeated movements of the Wasatch Fault Zone. The Wasatch Fault is seismically active and cuts alluvium and colluvium of Holocene age.

Geotechnical exploration included literature research, geologic mapping, seismic refraction, and core borings through the talus, bedrock, and fault zones (Figure 3). Lightweight drill rigs were flown in by helicopter and set up on wooden platforms. The platforms were constructed so as to minimize damage to the environment. The drill sites were restored to natural condition after drilling operations ceased. The drilling program began in early winter and could not be delayed because all drilling operations had to be complete by January 20 when golden eagles begin their nesting period on the rock cliffs above.

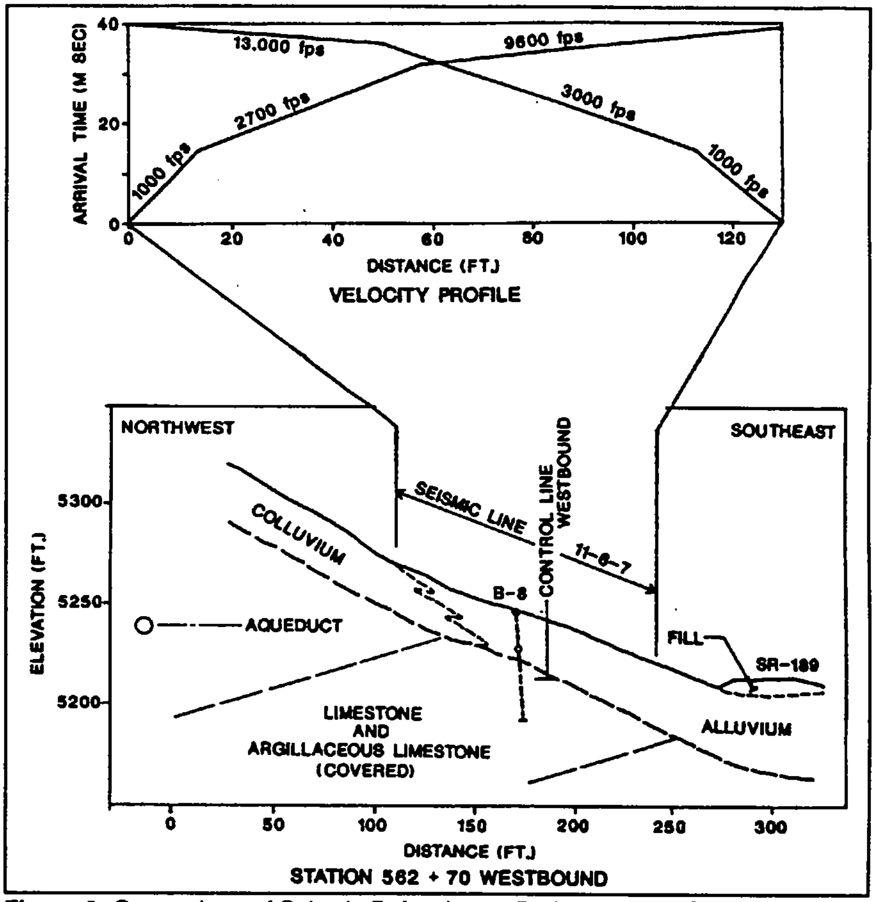

Figure 3: Comparison of Seismic Refraction to Borings; Provo Canyon Tunnel Project.

The Wasatch Range in the general area of the site is dominated by the Pennsylvanian aged Oquirrh Formation. More recent alluvial and colluvial deposits occur in the canyon bottoms and as a veneer covering the bedrock along the steep canyon slopes. The Oquirrh Formation consists predominantly of limestone and of limy to quartzitic sandstone.

Geologic units present along the alignment generally consist of the surfacicial talus, alluvial, and colluvial deposits and the Oquirrh Formation bedrock. The relationship of the geologic units is presented on the geologic cross section in Figure 4. A more detailed description of the geologic units is given in Chen-Northern (1991).

The talus is present throughout the project alignment at the base of most large outcrops. It forms steep, generally active slopes and is most abundant on the west end of the project. The talus is made up of loose gravel, cobble and boulder size material and becomes sandy and silty with depth. Talus slopes at the west end of the project are steep and slope material tends to move downhill. The base of the slopes have been steepened by road cuts for the existing highway. Colluvium covers a large portion of the site. The colluvium varies greatly in composition from silty sand to gravel with cobbles and boulders. The amount of fines within the colluvium typically increases with depth.

Bedrock exposed along the alignment is the upper approximately 800 to 1000 feet of the Bridal Veil Falls Member of the Oquirrh Formation. The rock is composed predominantly of limestone but contains argillaceous limestone beds and some sandy limestone beds. Beds range from thin shaly beds and argillaceous layers, to massive limestone where the thickness of beds is greater than 20 feet. The rock is predominantly gray to dark gray, but the sandy limestone layers weather to a pale yellowish brown. The limestone is fossiliferous and contains some chert nodules.

Attitudes of the beds just west of the alignment are near

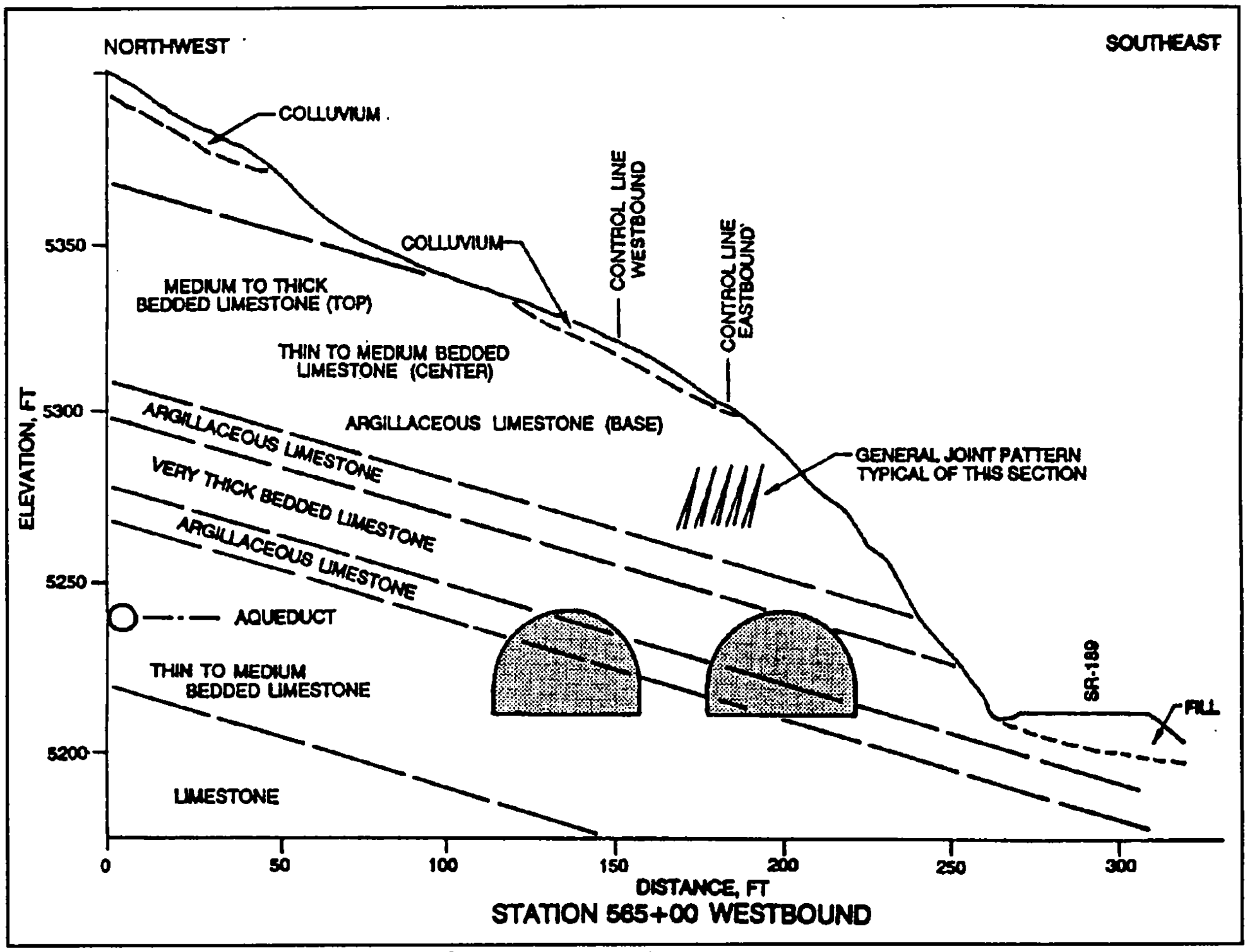

Figure 4: Geologic cross section; Provo Canyon Tunnel Project.

vertical. Beds dip on an average of 36 degrees to the east with a strike of North 15 degrees East in the western two-thirds of the project. Between the east anticline and syncline, beds dip approximately 18 degrees to the west with a strike of approximately North 8 degrees West. Beds at the extreme east end of the project dip approximately 30 degrees to the east with a strike of approximately North 3 degrees West.

Several small faults exist throughout the project with offsets of a few feet or less. Rock cores obtained during drilling also indicate the presence of brecciated and recemented zones. Of significance to the project is a relatively wide fault zone in the area of the eastern syncline. The zone is at least 120 feet wide where it is exposed in the road cut in this area. The rock is highly fractured and contains a large amount of brecciated and argillaceous rock.

Subsurface samples of the rock in the fault zone indicate that many of the fractures and much of the brecciated rock has been infilled with secondary calcite, improving the quality of the rock. The argillaceous limestone is shaly in portions of fault zones. Numerous slickensided discontinuities, which are not healed by secondary calcite, exist through this zone. A significant amount of shaly and argillaceous limestone was encountered within the fault zone.

Spacing of joints range from less than 1/4 inch to approximately 10 feet with an average spacing of 2 to 4 feet. The dominant joint sets are typically closed, slightly undulating, and range from smooth to rough. The less dominant joints are typically less continuous than the dominant set but are similar in character. Many of the joints have been recemented with calcium carbonate, but are weathered at the surface to an open joint.

This project is in the preliminary design stage. Changes in scope may result as the design evolves. Construction is presently anticipated in early 1992.

CONCLUSION

The tunnel projects described herein posed practical as well as technical challenges to deciphering the complex geologic conditions at each site. Methods adopted for geotechnical exploration were not ones ordinarily used on conventional highway projects. However, in the end, the usual geotechnical parameters were obtained for design and the projects proceeded uneventfully.

ACKNOWLEDGEMENTS

The projects described in this paper were designed by Parsons Brinckerhoff for the North Carolina Department of Transportation, U. S. National Park Service, Colorado Department of Highways, Hawaii Department of Transportation, Utah Department of Transportation, and Federal Highway Administration. Their permission to publish information relating to these projects is gratefully acknowledged.

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HIGH QUALITY ASPHALTIC CONCRETE PAVEMENT CONTAINING CHEMICALLY TREATED UNSOUND AGGREGATE

by

James R. Dunn, George M. Banino and Donald G. LeGrand

1.0 ABSTRACT

Unsound graywacke aggregate can be chemically protected from weathering deterioration in asphaltic concrete. This conclusion is the result of extensive laboratory and field tests. A group of polyelectrolytes potentially capable of producing semipermeable membranes were selected by General Electric (GE) and tested in the Dunn Geoscience Corporation (DUNN) laboratory. Polyalkylene imine (PEI-6), polyalkylene amine (PEIXD 305259.21) and polyalkylene polyamine (E-100) were applied in aqueous solution to a variety of aggregates and found to greatly reduce test losses in the magnesium sulfate soundness test and the New York State Department of Transportation (NYSDOT) freeze-thaw test on unconfined aggregate in 10% NaCl solution. The chemicals do not significantly alter the water absorption of the treated aggregates.

Approximately 1400 feet of asphaltic concrete containing untreated and E-100-treated unsound graywacke aggregate (71.8 to 88.1% magnesium sulfate soundness test losses) were laid in test strips on NYS Route 38B near Binghamton, New York, in June, 1982. One-half percent of a surfactant - phosphate ester of polyoxyalkylated fatty acid (Klearfac AA-270) - was added to the aqueous solution of the E-100 to improve the bond with the asphalt. After nine years the concrete with the treated coarse aggregate shows virtually no signs of deterioration whereas concrete made from untreated aggregate deteriorated significantly both by pitting of aggregate particles and by cracking. The E-100 continues to protect the aggregate even though the aggregate is worn down well below the surface of the original E-100 coating.

We conclude that asphaltic concrete pavements that contain E-100-treated coarse aggregate that has high sulfate-soundness or high NYSDOT freeze-thaw test losses may be significantly improved to better withstand deterioration related to frost action and winter salting. Although the aggregate used in the pavement tests is graywacke which is common in southern New York and central and western Pennsylvania, laboratory tests indicate that unsound carbonate rock aggregate could probably be similarly improved. The treatment of coarse aggregate by E-100 aqueous solutions should allow the use of aggregates which are not now acceptable because of high soundness test losses.

2.0 INTRODUCTION

2.1 Purpose

The purpose of this research was to find a method to improve the soundness of mineral aggregate with chemical agents which were relatively inexpensive, water based, and easily applied.

2.2 Rationale, Soundness

The action of ice as a cause of unsoundness in many New York aggregates was found to be far less important than warming and cooling and/or wetting and drying of aggregates with water, especially if the water contained dissolved salts (Dunn and Hudec, 1965A, 1965B, 1966, 1968, and 1972 and Hudec, 1965). One test, the 40-year old NYSDOT 25-cycle freeze-thaw test of unconfined aggregate in 10% NaCl solution, is intended to determine how aggregate holds up under conditions of normal winter highway salting. We decided that if aggregate could be protected from salts, perhaps, the aggregate could be made more sound.

The search for coating materials was restricted to chemicals which would allow the free movement of water but retard the movement of salts. We felt that if water vapor were unable to escape, a back pressure caused by trapped water vapor could weaken the bond between the rock and the cementitious matrix (akin to the blistering of paint on the sunny side of a house).

2.3 Nature of Test Program

In 1981 a cooperative program between GE and DUNN was started to find suitable chemical coatings. The chemicals were selected by GE and they were then applied to aggregates and tested by DUNN in its laboratory. In all, about 30 surfactants and polymers were tested in various solvents. Over 200 physical tests of aggregate were conducted in the DUNN laboratory. Certain other tests to determine: 1) the stability of the chemicals at high temperature and under simulated weathering and, 2) the penetration of chemical coatings and of salts were conducted in the GE laboratories. For this report only those test results which are most relevant to the chemicals actually selected for field testing are summarized.

3.0 CHEMICAL AND PHYSICAL CONSIDERATIONS

3.1 Background, Polymer Chemistry

3.1.1 Macromolecular Polyelectrolyes

The distinction, in classical chemistry, between uncharged molecules and electrolytes composed of ions has its counterpart in the field of high polymers. The attachment of large numbers of ionized species produces striking modifications in the polymer characteristics. Many natural products or their derivatives such as gelatin, alginic acid, and various de-esturified pectins are polyelectrolytes. Polymethacrylic acid, polyethylene imine, polyvinyl pyridine, and polyacrylimide are examples of synthetic polyelectrolytes. Those polyelectrolytes which possess negatively or positively charged surfaces are called, respectively, anionic and cationic while those containing both are called ampholytes, and the ionized macromolecular polyelectrolyte is called a "polyion".

3.1.2 Gelation and Coagulation of Polyions

Ionized polyelectrolytes attract and bind with multivalent ions whose charges are of opposite sign to that of the polyelectrolyte. Further, the state of the system depends upon the number and type of the multivalent ions and the number of ionized groups on the polyelectrolyte. More specifically, the addition of the multivalent ions leads to the formation of clusters in which some of the multivalent ions are linked to two or more polyions. At a certain point, found by experiment to be quite sharp, the clusters become large enough to precipitate out and become visible; this is known as the "gel point". Alternatively, the total system will transform into a soft, rubbery material similar to gelatin and is also referred to as a gel. In either case, the basic chemical mechanism of this interaction of polyions with the multivalent ions remains the same.

3.1.3 Adsorption/Absorption Process

When nonporous materials such as glass or metal plates are immersed in solutions containing high polymers, the long chain molecules are adsorbed on the surface to minimize their free energy. The energies of this process involve Van De Waals forces, electrostatic forces, and chemisorption as well as free energy changes in the molecules themselves due primarily to entropic configurational and conformational changes. If the adsorbed species are polyions, then some of the ions may chemically interact with the surface while others will be free to react with the surrounding environment.
Similarly, if a porous structure is immersed in solutions containing high polymers, depending upon the size and connectivity of the pores as well as the time of contact, the polymer molecules will be adsorbed by the structure. A necessary consequence of this process is the adsorption of some of the species on the internal surface of the structure for the same reasons described above. The remaining ones will form an internal bulk phase. Again, as described above, if the absorbed species are polyions, either gelation or coagulation may result.
As a result of the process, a semi-permeable barrier will form.

3.1.4 Semi-Permeable Membranes and Salt Rejection

Polyelectrolyte membranes are known to be perm selective. Their selectivity is due to Donnan exclusion of salt from the membrane phase. This represents one class of materials which might be used for the desalination of brackish water and sea water.

In simple physical terms, when the polyelectrolyte is gelled or coagulated through the use of appropriate divalent or trivalent counterions, electroneutrality results. The driving force for the diffusion of ions through such a membrane is predominantly the concentration gradient, which is opposed by the condition for electroneutrality and the effects of viscosity.

Undissociated water is without charge and may move freely through the membrane without altering its electroneutrality. The rate of movement will only involve classical intensive parameters of temperature, pressure, and concentration.

For a more complete description of the above sorptive mechanisms see Encyclopedia of Polymer Science and Technology, 1969, v. 10, pp. 781-761, Helfferich (1962), Lakshminarayanaiah (1969) and Moorse (1955).

3.2 Mechanisms of Rock Deterioration and Testing

Rocks, as with all other materials including metals, ceramics, and polymers, may undergo environmentally-created brittle failure. The mechanisms of such phenomena are not fully understood, although useful working hypotheses have been developed.
On the basis of visual and microscopic examination of the deterioration of rock aggregate in DUNN's laboratory, two hypotheses were developed: first, argillaceous rocks that absorb H2O also dissolve and reprecipite inorganic salts. This may result in a disturbance of the mechanical equilibrium of the structure and lead to the formation of a new structure which is at mechanical equilibrium with respect to the total structure, but may cause local overstress. For example, a piece of ordinary plate window glass, covered on one side by an ultra thin (~500 Å thick) coating of Ag or Au will have Na ions on the untreated side replaced by K ions when placed in a molten bath of K2NO3 at 600°C. When the plate glass is removed from the bath and cooled to room temperature, it will warp. While the system will be in mechanical equilibrium, the concave side of the glass will be in a state of biaxial tension, which, because of local inhomogeneities, may undergo spontaneous failure either by spalling (conchoidal failure) or by a dicing of the total sample. Aggregate rock, due to both its complex geometry, homogeneous composition, and structure, may behave in a similar way.

An alternate hypothesis is that the deterioration of rock containing clay, even minor clay, may be subject to the same water sorption phenomenon which causes shales to slake. For reasons not fully understood, many slightly argillaceous rocks deteriorate more rapidly when alternately frozen and thawed in NaCl-rich aqueous solution, even though the salt greatly reduces the amount of ice which forms, in some cases to zero (Dunn & Hudec, 1972). The deterioration of many argillaceous rocks upon wetting and drying, in addition to accelerated freeze-thaw deterioration in the presence of NaCl, may be related to an order-disorder phenomenon of water layers on the surface of clays. The water layers may expand and contract with wetting and drying, with changes in humidity, or with warming and cooling in the presence of water. The presence of inorganic ions may increase the thickness of the water layers increasing the stress on the walls of rock pores.
Because of such theoretical uncertainties, empirical laboratory tests were conducted in the DUNN laboratory to determine soundness of treated and untreated rock.

3.3 Chemicals Tested

The most effective of the polyelectrolytes tested (Table 1) were polyalkylene imine (PEI-6), the polyalkylene amine (PEIXD 305259.21), and polyalkylene polyamine (E-100). The chemicals tested were dissolved in water and precipitated on the aggregate by wetting and evaporation in an oven at 230°F.

4.0 LABORATORY TESTS

4.1 Application of Chemicals

The aggregate was dipped into the various solutions of polyelectrolytes at various concentrations at room temperature and then the wet aggregate was dried at 230°F for two to three hours.

4.2 Absorption Tests

Water absorption tests were conducted according to ASTM C 127-80 procedures to determine the extent to which water (pure and NaCl solutions) could move through the coatings precipitated from various solutions. Table 3 summarizes the test results on a graywacke aggregate.

Additional absorption tests of dilute aqueous solutions of NaSO4, MgSO4 and Na(OH)2 on treated graywacke and of absorption of pure water on untreated graywacke aggregate all showed similar results; the percent of water or solution absorbed by graywacke is little influenced by treatment with polyelectrolyte solutions.

4.3 Energy Dispersive X-ray Analysis (EDX)

In order to determine the depth of penetration of the

Table 1

POLYELECTROLYTES TESTED FOR POTENTIAL FOR UPGRADING AGGREGATES.

Chemical	Trade Name
Polyethylene imine	PEI-6, Polymin 6
Poly acrylic acid	PAA
Poly acrylamide	
Poly azideridine	XMMA-7
Poly ethanolamine	
Poly functional amines	Jeftamines
Copolymers of methyl	Gantrez
Vinyl ether and maleic anhydride	
Diethylene triamine	DETA
n-Proplylamine	
Polyalkylene polyamine	E-100
Polyalkylene amine	PEIXD 305259.21

TABLE 2

WATER ABSORPTION, TREATED AND UNTREATED GRAYWACKE AGGREGATE

Treatment	Solution	Percent Absorbed
None 25% E-100	5% NaCl 5% NaCl	2.0 1.6
None None 10% E-100	Water 3% NaCl 3% NaCl	1.9 1.8 1.7
None None	Water 3% NaCl	1.9 1.8
10% E-100 10% E-100	Water 3% NaCl	1.6 1.7
25% E-100 25% E-100	Water 3% NaCl	1.5 1.4
None 20%, PEI-6	Water Water	1.8 1.7

polymer into the porous rock surface as well as the extent to which the polymer prevented the inward migration of metallic ions, graywacke aggregate particles treated with a 10% solution of E-100 were immersed in 10% solutions of magnesium sulfate, copper sulfate, and zinc sulfate. The samples were then fractured and EDX in a scanning electron microscope was used to study the fracture surface. Based on metallic ion concentrations the polymer appeared to have permeated the rock approximately 1 um and that the depth of penetration was not signficantly increased by longer immersion times. Metallic ions were concentrated on the outside of the graywacke.

4.4 Magnesium Sulfate Soundness Test Results

The procedure for the ten cycle NYSDOT magnesium sulfate soundness test (New York State Test Method 207 B-76) for concrete aggregate is similar to that described in ASTM C 88-76, Soundness of Aggregate by Use of Sodium Sulfate or Magnesium Sulfate. In New York the maximum allowable 10-cycle magnesium sulfate soundness test loss for coarse aggregate for portland or bituminous cement concrete use is 18%. Table 3 is a partial list of results for treated aggregate tested as above. Each result is compared to a reference test on another sample of the same aggregate untreated.
The tests demonstrated that: 1) aggregates of several types were protected from deterioration in the magnesium sulfate soundness test and 2) the amount of protection generally increases with the concentration of the chemicals in the aqueous solutions.

4.5 Freeze Thaw Tests

The NYSDOT's freeze-thaw test has been used for about 40 years to help evaluate the soundness of New York's aggregate. The test is unique in that the unconfined aggregate is frozen and thawed in a 10% NaCl solution (for details, see NYSDOT Test Method 208-76). The reason for the salt is that the test is meant to simulate the exposure of aggregate to road salts during winter conditions. As previously noted, this NYSDOT test greatly accelerates rock deterioration even though the amount of ice which forms in the frozen rock is far less than when pure water is used. The maximum allowable loss for portland and bituminous cement concrete aggregates is 10%.

Table 4 below summarizes the results of freeze-thaw tests on PEI-6-treated aggregate. The PEI-6 treatment appears to protect both aggregates from alternate freezing and thawing cycles in a 10% NaCl solution.

5.0 ASPHALTIC CONCRETE ROAD TEST

5.1

The NYSDOT suggested that a road test be conducted using asphaltic concrete containing both treated and untreated aggregate. The chemicals were found to be stable at 500°F and, hence, it was felt that they should survive the asphaltic concrete manufacturing process.

5.2

The aggregate selected was the unsound graywacke sandstone from a quarry in western New York. The untreated stone had 10-cycle magnesium sulfate soundness test losses of over 70%. The polyalkylene polyamine, E-100, was selected as the semipermeable membrane. After considerable experimentation using various anti-stripping agents, phosphate ester of polyoxyalkylated fatty acid (Klearfac AA-270) was selected to enhance the bond between the asphalt and the aggregate.

5.3

NYSDOT gradations 6A and 7A (see Table 5) were immersed in a 20% solution of E-100 plus 0.5% Klearfac AA-270 in a front end loader bucket with two holes in the bottom through which the solution could drain. The aggregate was then stockpiled over the weekend.

5.4

The 4-inch thick strips were laid down on Route 38B, Broome Co., about 10 miles west-northwest of Binghamton, New York, during June, 1982 (Figure 1). The two adjacent treated strips, one with 6A and one with 7A aggregate, were 790 feet long; the two adjacent strips of asphaltic concrete with untreated 6A and 7A aggregate were 625 and 610 feet long, respectively. The untreated and treated test strips were separated by almost 1,000 feet of asphaltic concrete with untreated aggregate. Figure 2 shows details of the test strip layout.

TABLE 3

TEN-CYLCE MAGNESIUM SULFATE SOUNDNESS TEST LOSS ON AGGREGATES FOR VARIOUS CONCENTRATIONS OF CHEMICALS COMPARED WITH THE SAME AGGREGATE UNTREATED

Aggregate Type	Chemical Treatment	% Solution Concentration	% Soundness Loss Untreated	Treated
Graywacke	PEIXD 30259.21	5	71.8	4.3
Graywacke	PEIXD 30259.21	10	71.8	4.1
Graywacke	PEIXD 30259.21	20	88.1	0.6
Graywacke	E-100	20	88.1	0.6
Graywacke	E-100	25	87.9	0.5
Argill.Do1	E-100	10	30.7	1.3
Argill.dol	PEIXD 30259.21	5	30.7	4.2
Argill.dol	PEIXD 30259.21	25	56.9	1.2
Argill.dol	PEI #6	10	56.9	4.3
Argill.dol	PEI #6	30	33.3	2.4
Marble	PEI #6	5	22.3	16.2
Argill.dol	PEI #6	1	74.0	72.0
Argill.dol	PEI #6	10	74.0	2.4
Shaley gravel	PEI #6	5	68.0	51.2
Shaley gravel	PEI #6	25	68.0	19.1
Cherty gravel	E-100	25	54.6	16.3
Shaley gravel	E-100	10	29.7	14.7

NOTE: The aggregates tested were from commercial producers. The names of the producers are not disclosed because of the need to keep confidential information about quality. The graywacke was typical of occurring throughout southwestern New York and northern Pennsylvania both as bedrock and as a component of gravels of the area. The argillaceous dolomites contained disseminated clays with the clays at the boundaries of dolomite crystals (the rejection texture of Dunn and Hudec, op cit).

TABLE 4

TEST RESULTS FOR TREATED AND UNTREATED AGGREGATE SUBJECTED TO FREEZE-THAW TEST, NYSDOT TEST METHOD 208-76, IN 10% NACL SOLUTION.

Aggregate Type	Solution Concentration	Percent Loss Freeze-Thaw Test Untreated	Treated
Argillaceous Dolomite	25	28.5	7.9
Graywacke	20	50.8	10.0

TABLE 5

NO. 6A AND 7A AGGREGATE SURFACE COURSE GRADATIONS AND ASPHALT CONTENT OF ASPHALTIC CONCRETE CONTAINING TREATED AND UNTREATED GRAYWACKE AGGREGATE.

Sieve Sizes	Pecent Passing 6A	7A
1"	100	100
1/2"	90-100	90-100
1/4"	65-85	90-100
1/8"	36-65	45-70
20	15-39	15-40
40	8-27	8-27
80	4-16	4-16
200	2-6	2-6
Asphalt, Content, %	5.8-7.0	6.0-7.0

5.5

Approximately 522 feet of the test strips were sprayed about five months after the paving with a 20% solution of E-100 to determine if such surface application might have any measurable effect (see Figure 2 for locations).

5.6 **Observations**

5.6.1 After the first winter no differences were observed by NYSDOT engineering geologists and engineers with all test strips holding up well. NYSDOT personnel suggested that the lack of any visible deterioration was caused by the very careful procedures for making and laying the asphaltic concrete.

5.6.2 In the second year in May, 1984, after two winters, NYSDOT personnel observed significant differences in the pavements. The number of deteriorated particles showing up as pits was distinctly less in the asphaltic concrete strips made with treated aggregate than in the strips made with untreated aggregate. Although no precise measurements were made, the ratio was estimated in the field as approximately one pit (treated) to 5 pits (untreated).

Four 4-inch diameter cores, about 4 inches long were cut by NYSDOT for further study in the laboratory. Two were from the untreated and two from the treated pavement. Laboratory observations in the DUNN laboratory verified the field observations: the amount and severity of the pitting appeared to be much greater for the untreated pavement. The differences in the amount of pitting were not quantified.

A modified magnesium sulfate soundness test was conducted on the treated and untreated cores. Graywacke in the treated core deteriorated less than the graywacke in the untreated core. This was even though the exterior treated layer of the aggregate particles had been cut by the sawing and the presumably untreated interior of the aggregate exposed.

5.6.3 The asphaltic concrete test strips were again observed in March, July and August of 1990 and, finally, in April, 1991. The spray-treated portions of the asphaltic concrete pavements were apparently not improved by the treatment. However, test strips of asphaltic concrete containing treated aggregate were in excellent shape with only minor local cracking or wheelpath cracking. The test strips which were made with asphaltic concrete containing untreated aggregate showed significant wheelpath and other cracking (see figures 3, 4, 5, and 6). The test strip with the finer 7A untreated aggregate was significantly more deteriorated than that containing the untreated 6A but all sections of both gradations had wheelpath cracking.

Figures 7, 8 and 9 are typical of the asphaltic concrete with treated aggregate. The amount of cracking is distinctly more for the concrete with untreated aggregate than for the treated aggregate. Figure 9 is of a rare cracked portion of pavement with treated aggregate.
Pitting of the 6A (coarser) aggregate is clearly observed in the field but very difficult to show in photographs. Photographs of dry and partially wetted pavement are of only limited value for quantifying the amount of pitting. Because of the considerable danger of studying the surface of a busy highway, six peels of latex-type material were taken so the surface textures could be studied in a laboratory environment. A section of the outside wheel track of the asphaltic concrete with the 6A aggregate was first sprayed with a common vegetable oil cooking spray and then a thin layer of latex painted on a little over a square foot. After drying, a piece of cheesecloth was laid on the painted area and another coat applied. Two more coats were applied and dried before the peel was removed. The process took about two hours. Three peels of typical treated pavement and three peels of typical untreated pavement were taken. The peels were approximately 12 feet apart.

The peels of the untreated pavement were much rougher than were those of the treated pavement (see figures 11, 12, 13 and 14). The degree of roughness was determined by making four one foot traverses across each peel for a total traverse of 12 feet of treated and 12 feet of untreated pavement peels. The traverse strips were a half inch wide and approximately 2½ inches apart. Protrusions (pits in the pavement) which were from 0.05 to 0.10 inches high and those over 0.10 inches were counted. The peel from untreated pavement had 46% more protrusions in the 0.05 to 0.10 inch size (70 versus 48). There were 850% more protrusions in the over 0.10 inch size range (19 versus 2). Although we feel that the percentage differences are based on too few feet of counting to be considered precise, the categories of the numbers are considered to be significant and are consistent with field observations.

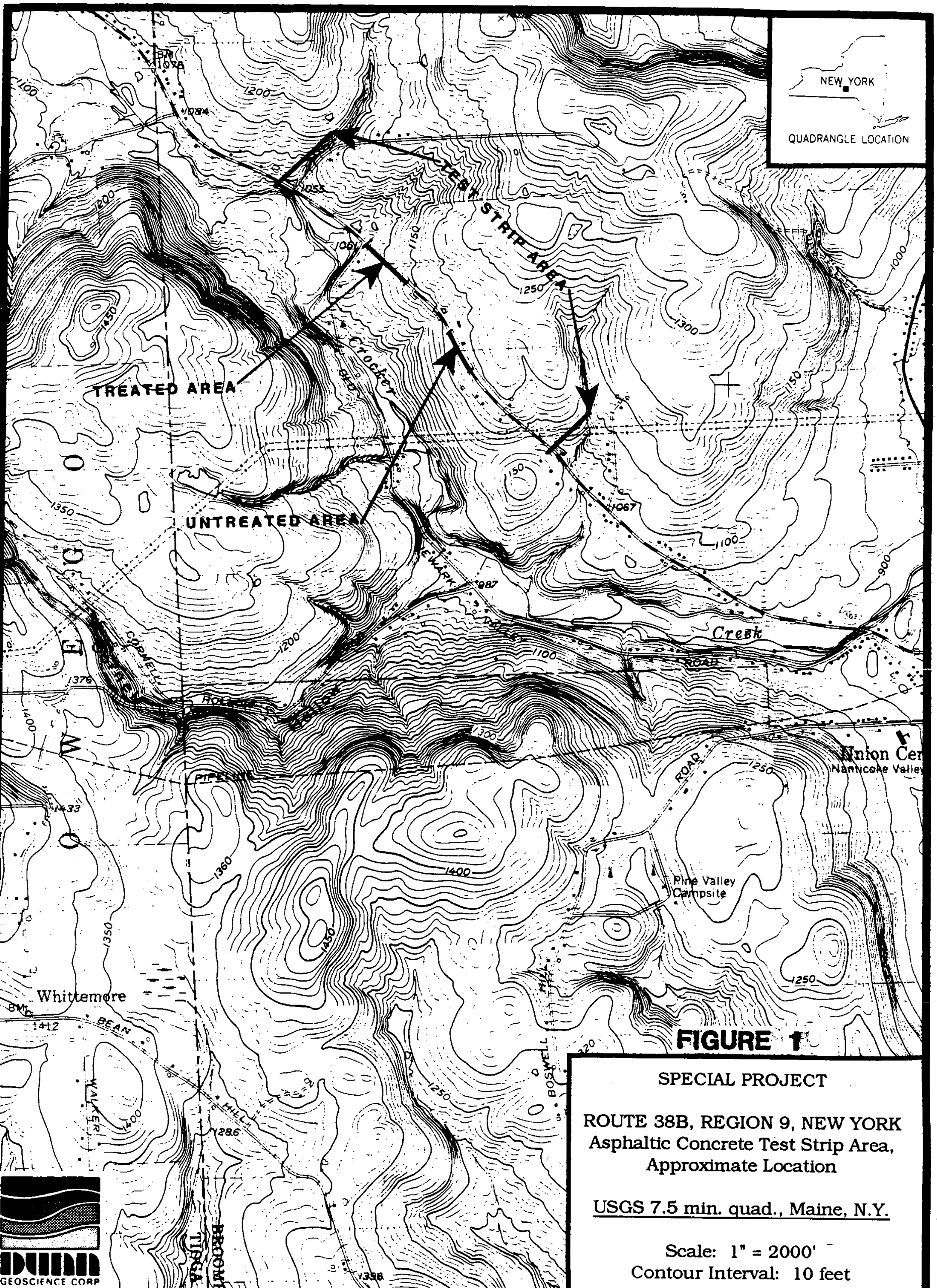

FIGURE 1

SPECIAL PROJECT

ROUTE 38B, REGION 9, NEW YORK
Asphaltic Concrete Test Strip Area,
Approximate Location

USGS 7.5 min. quad., Maine, N.Y.

Scale: 1" = 2000'
Contour Interval: 10 feet

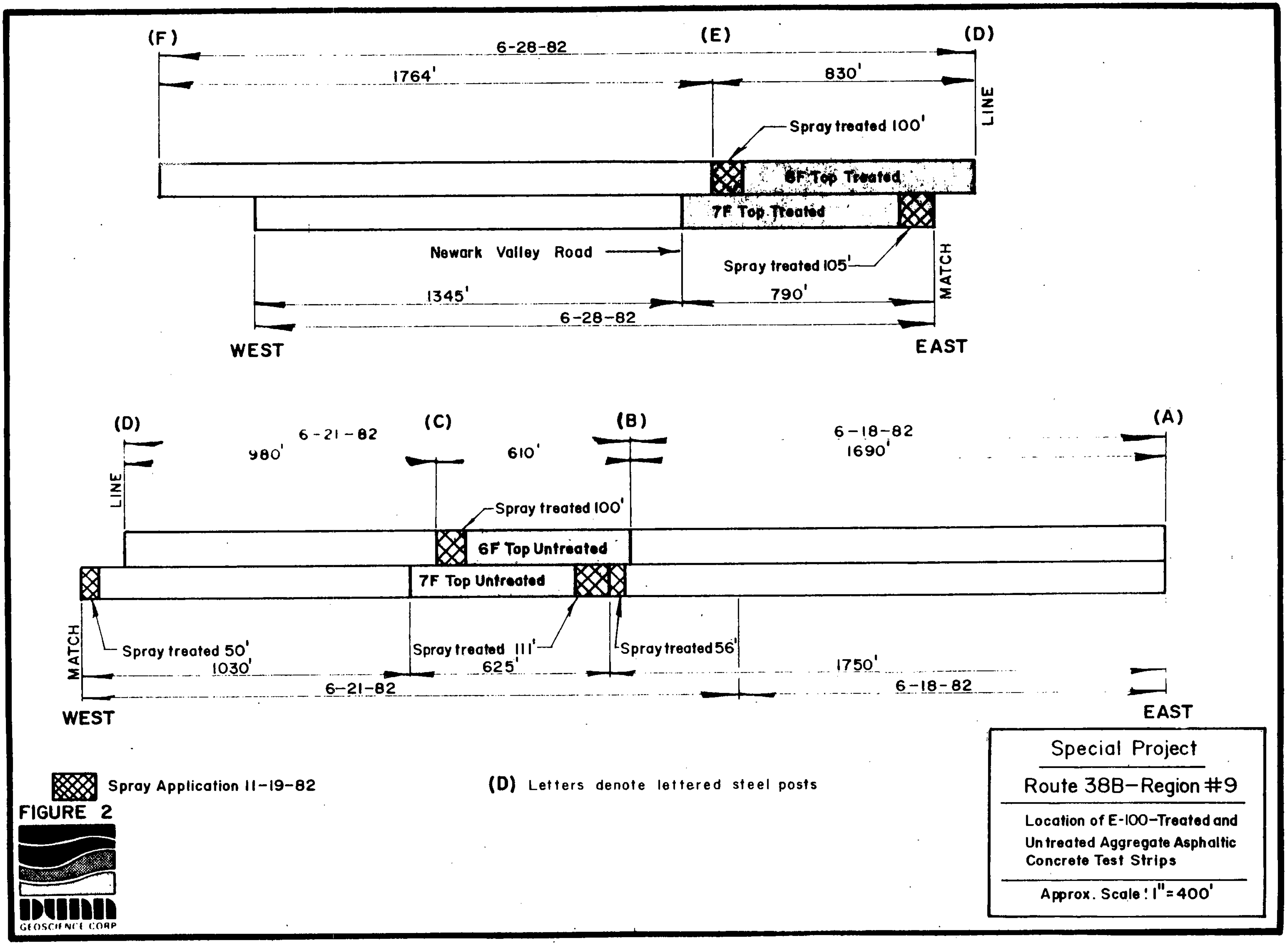
(F)
(E)
(D)
6-28-82
1764'
830'
LINE
Spray treated 100'
6F Top Treated
7F Top Treated
Newark Valley Road
Spray treated 105'
MATCH
1345'
790'
6-28-82
WEST
EAST
(D)
(C)
(B)
(A)
6-21-82
980'
610'
6-18-82
1690'
LINE
Spray treated 100'
6F Top Untreated
7F Top Untreated
MATCH
Spray treated 50'
Spray treated 111'
Spray treated 56'
1030'
625'
1750'
6-21-82
6-18-82
WEST
EAST
Spray Application 11-19-82
(D) Letters denote lettered steel posts
Special Project
Route 38B–Region #9
Location of E-100–Treated and Untreated Aggregate Asphaltic Concrete Test Strips
Approx. Scale: 1" = 400'
GEOSCIENCE CORP

FIGURE 2

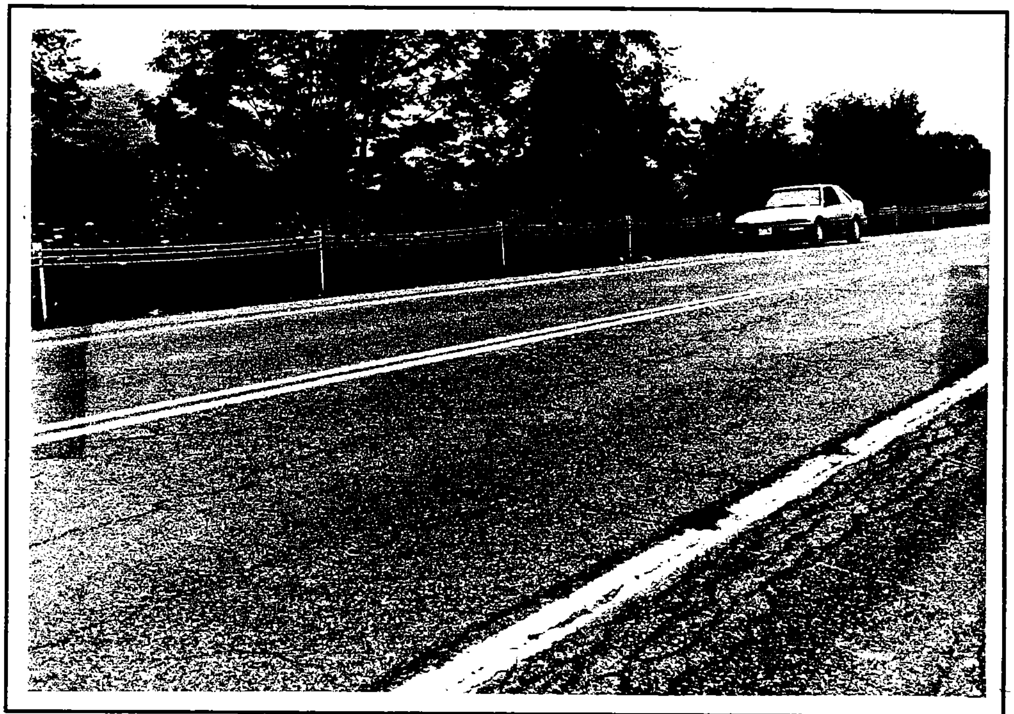

Figure 3: Typical pavement with untreated aggregate, 6A gradation foreground, 7A gradation in background (the shoulder was not part of the test).

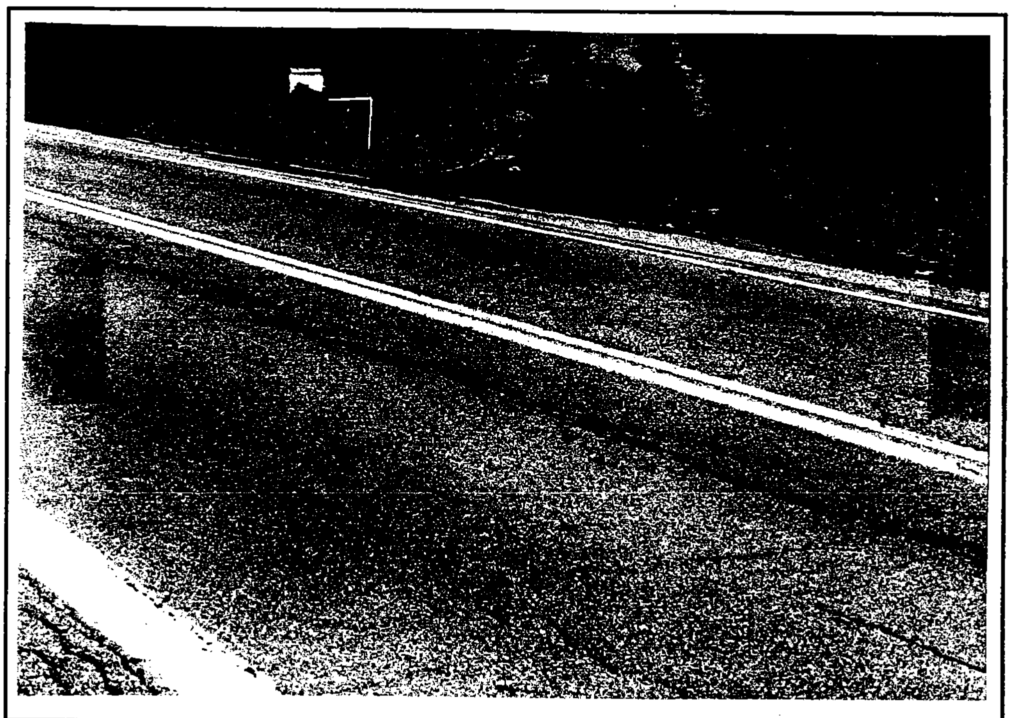

Figure 4: Pavement with untreated aggregate, 7A gradation foreground, 6A gradation in background, showing wheelpath cracking.

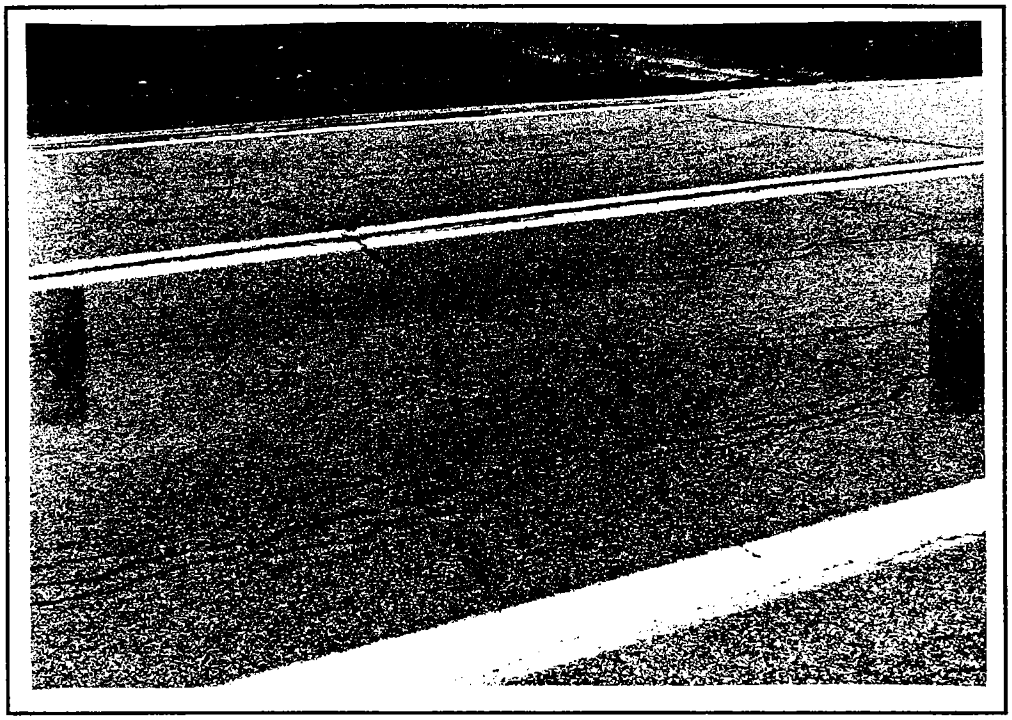

Figure 5: Pavement with untreated aggregate 7A gradation foreground, 6A background; wheelpath and other cracking.

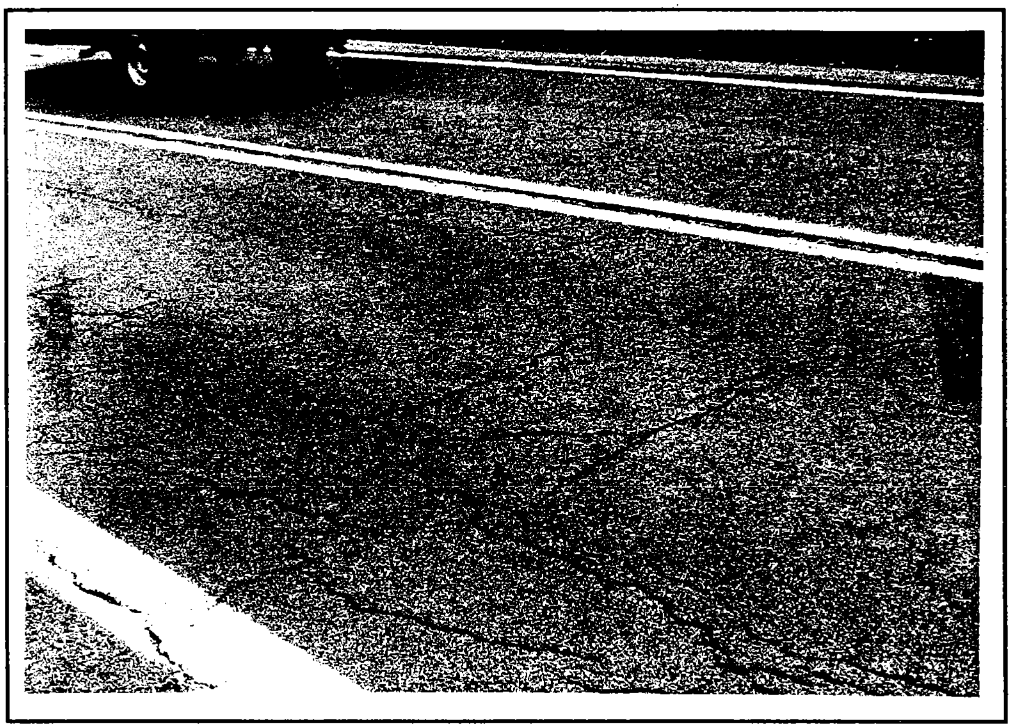

Figure 6: Pavement with untreated aggregate, 7A gradation in foreground, 6A background; wheelpath and other cracking.

Figure 7: Typical pavement with treated aggregate, 6A gradation in foreground, 7A gradation background. Test pavement starts adjacent to the automobile on the right.

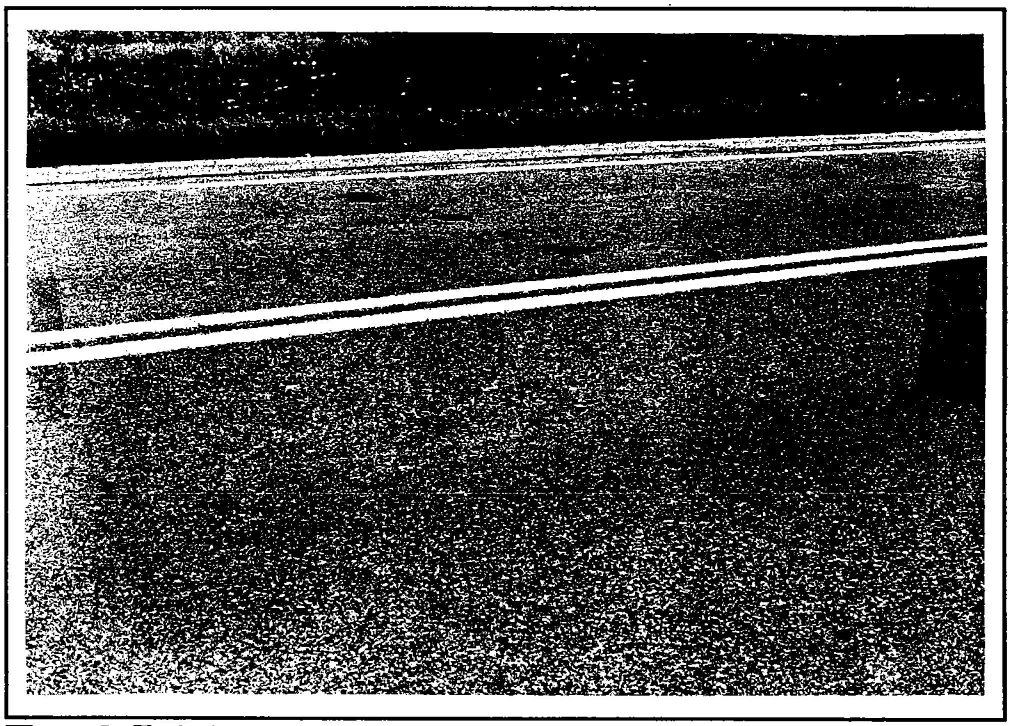

Figure 8: Typical pavement with treated aggregate, 7A gradation foreground, 6A gradation background.

Figure 9: Section of pavement with treated aggregate, 7A gradation in foreground, 6A gradation in background showing some longitudinal and other cracking. This is the poorest part of the concrete with treated aggregate.

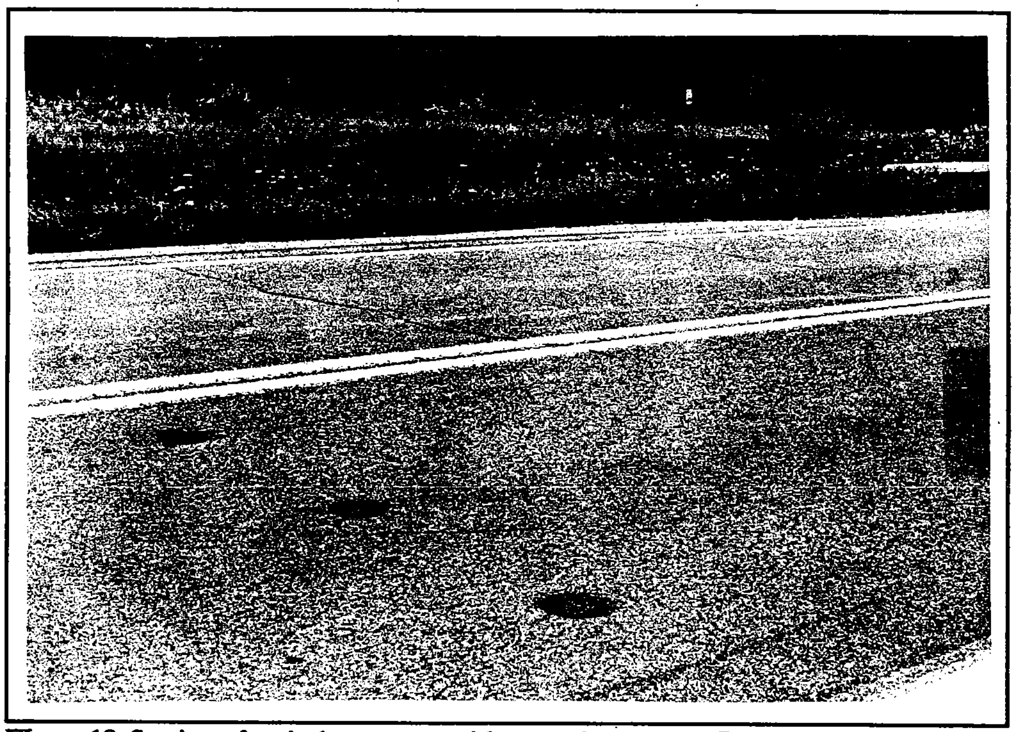

Figure 10: Section of typical pavement with treated aggregate, 7A gradation foreground, 6a gradation background.

6.0 DISCUSSION

6.1

Water adsorption and EDX demonstrate that water passes through E-100 and PEI-6 coating on graywacke aggregate while metallic ions do not penetrate the coating.

6.2

The results of the magnesium sulfate soundness tests and the freeze-thaw tests in 10% NaCl solution show that, as intended, the chemicals used reduce significantly the amount of degradation of aggregate in the tests.

6.3

The improvement of asphaltic pavement with E-100-treated graywacke aggregate over untreated aggregate is easily observed in the field. The more sound nature of the treated aggregate is clearly indicated by the much less amount of pitting of the aggregate particles in the 6A top course. In addition, major wheelpath and other cracking occurs in the concrete containing untreated aggregate, but cracking of the surface is very minor for the concrete with treated aggregate.

6.4

The significant wheelpath cracking of the asphaltic concrete containing untreated graywacke aggregate may be related to the greater deterioration of the aggregate. It is probable that the surfactant Klearfac AA-270 improves the bond between the aggregate and the asphalt and may be partly responsible for the greatly reduced wheelpath cracking of the asphaltic concrete with treated aggregate.

6.5

The surface of the aggregate is significantly worn down by tire abrasion on all of the treated asphaltic concrete, but the aggregate still appears to be protected. This seems to be consistent with the magnesium sulfate soundness test conducted on cores of asphaltic concrete with treated and untreated aggregate in which the treated aggregate fared better even though the cutting of the core exposed the untreated interior of aggregate particles.

6.6

Although the authors are not able to fully explain all of the phenomena observed, it appears that protecting unsound aggregate from salts while allowing water to move freely through a surficial chemical membrane can significantly improve the aggregate and improve asphaltic concrete surface courses for up to nine years.

6.7

The graywacke aggregate tested was from a quarry in New York. This graywacke is similar to graywackes occurring throughout southwestern New York and western Pennsylvania both as a bedrock and in gravels. It appears likely that many marginal to submarginal sources of aggregate in southwestern New York and

6.8

central and western Pennsylvania can be turned into viable sources for use in asphaltic concrete by treating the aggregates with E-100. Although only graywacke was tested in the asphaltic concrete pavements, other aggregate types such as argillaceous dolomites or limestones might also benefit from such treatment. In fact, it appears possible that any aggregate which has high losses in tests involving sulfate, sodium chloride or other salts could benefit from E-100 treatment.

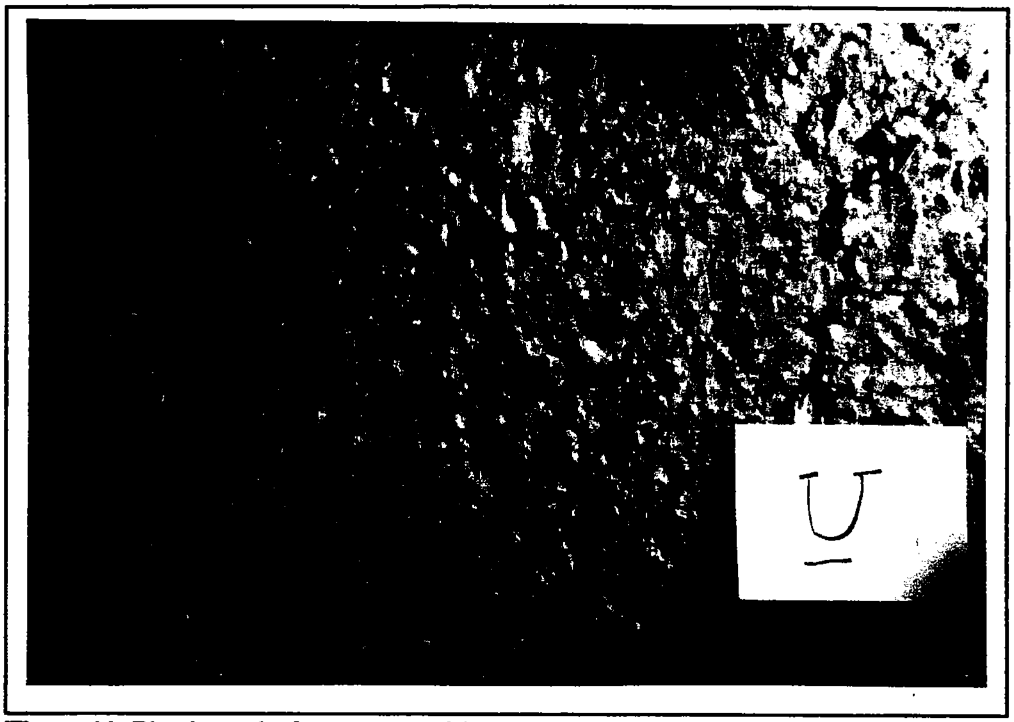

Figure 11: Plastic peel of pavement with untreated 6A aggregate, obliquely lighted.

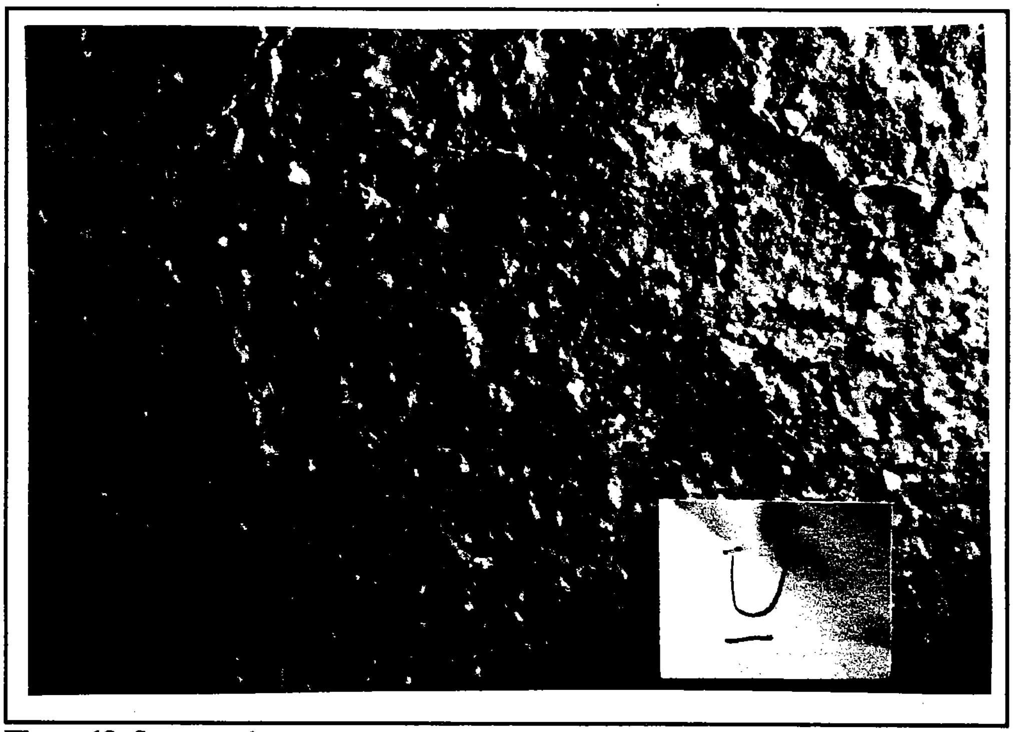

Figure 12: Same as above.

Figure 13: Plastic peel of pavement with treated 6A aggregate, obliquely lighted.

Figure 14: Same as above.

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IMPROVING AGGREGATE QUALITY BY CHEMICAL TREATMENT

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ABSTRACT

Many fine-grained aggregate types such as argillaceous carbonates, volcanics, sandstones, and chert and shale impurities degrade with repeated wetting and drying and with freezing and thawing, especially under the influence of de-icing salts. **Surface** coatings to prevent water and/or salts from entering the aggregate pores have been found successful, but expensive.

Chemical treatment of the **internal** pore surfaces by surface-active chemicals has proven effective in reducing aggregate (and concrete) deterioration. Large, electronegative anions such as phosphates and nitrates interfere with the adsorbed water layer and adsorbed de-icing salt ions (Na), reducing osmotic pressure differential between the micro pores and larger pores. Large cations, such as those found in ammonium salts compete with Na ions for internal surface sites, and effectively disrupt the adsorbed water layer. Lowering the osmotic pressure reduces the cracking of the aggregate and concrete. Several chemicals were found to be effective in pre-treatment of aggregates, and as admixtures to de-icing salts, significantly reduced the aggregate and concrete deterioration.

It is possible to pre-treat the aggregate prior to its use in concrete, treat the concrete, or mix the chemicals with de-icing salts; all of these methods significantly reduce the deterioration of concrete surface. Laboratory test results that show the effectiveness of the various treatments are presented in the paper. The poorer the aggregate, the greater is the beneficiating effect of chemical treatment.

INTRODUCTION

The use of de-icing salts (largely NaCl) in the northern latitudes of North America has resulted in progressive deterioration of the concrete road infrastructure, especially the bridges. To minimize the damage, steps have been taken and specifications issued. These specifications were designed to make either the cement paste, the mortar, or the aggregate (or concrete as a whole) less prone to deterioration. Concrete paste can be made resistant to de-icing salt by air entrainment. However, the aggregate that makes up roughly 70% of the concrete must be generally accepted as is--after careful testing, and possibly physical beneficiation to remove the more harmful particles. The testing is not always sufficient to recognize the deleterious aggregate, and the conditions of testing are not specifically designed to simulate long-term exposure to and progressive saturation with de-icing salts.

The sources of aggregate with good service record are getting depleted, and new, sources of acceptable aggregate are difficult to find and to develop, especially in the more urbanized regions that are the largest users of the material. Two alternatives are available to make better use of scarce resources and funds: make the concrete last longer, or use the material available, even if it is of lower quality. The two alternatives seem diametrically opposed. New technology is needed to bring them closer together.

Physical beneficiation as means of improving the quality of the aggregate has been used successfully for years. It is based on the observation that most deleterious aggregate fragments are generally of lower specific gravity than the bulk of the 'good' aggregate. Heavy liquid suspension is the most common method of 'floating' and separating the deleterious fraction. The method works on such materials as chert, shale, and weathered rock fragments. It is ineffective against the more subtly deleterious particles, such as very fine-grained and argillaceous fragments of many rock types, especially carbonates.

Surface coating of aggregate by polymers and oils (Cady et al., 1978, Ohama et al., 1987) has been suggested as a means of beneficiation. The surface coating, to be effective, should not affect the bonding properties of aggregate and should, preferably, be permeable to water but not to salt--a molecular sieve.

The beneficiation suggested in this paper is a chemical treatment of internal surfaces of rock aggregates to make them less attractive to Na^+ and other cations thought responsible for much of the deterioration. The freeze-thaw deterioration is accelerated by the de-icing salts--a seeming contradiction. The explanation of the deterioration process under de-icing salts and means of preventing it are discussed in the next section.

THEORY OF DE-ICER DAMAGE

A classical paper on the relationship between the physical properties and the frost resistance of aggregates was written by Verbeck and Landgren in 1960. Since then, it was found that the grain size, pore size, and total internal surface area of rock aggregates has a major influence not only on freeze-thaw durability, but also on wetting-drying resistance and overall durability of the aggregate. As the grain size decreases, the total surface area of the same mass of solid increases. No exact mathematical model can be used, since grains are irregular in shape. For illustration purposes, three shape end members could be considered: a cube, a square, and a platelet (flat rectangle). Table 1 shows how a unit mass of solid increases in surface area as the particle size decreases.

Table 1 is graphically represented in a log-normal diagram of Figure 1. Note that the particle shape becomes less important as the particle size decreases--i.e., the total surface area is governed more by grain size than by grain shape. Internal surface area of a rock composed of fine particles plays a major role in aggregate and concrete deterioration (Hudec, 1989).

The physical breakdown observed in diverse materials is strikingly similar, starting with the development of microfractures and cracks. These cracks may lead to complete disintegration of the material. Deterioration occurs only in the presence of water. Pure water was shown to be innocuous; it is the nature and the concentration of dissolved ions in the water that dictate the severity of deterioration. The general scenario of deterioration involving dissolved ions was summarized by Hudec (1989) and can be outlined as follows:

The pore surfaces possess residual charges, usually negative, the strength of which is the function of the degree of disorder in the crystal lattice of the minerals making up the surface. The surfaces attract the ions dissolved in pore water, concentrating them in layers along the surface. The small pores thus contain a higher concentration of ions relative to a larger pore; an osmotic differential may develop with attendant expansive and contractive forces. The expansive forces, acting against the tensile strength of the solid, may microfracture the solid, creating more fine pores.

Table 1. **SURFACE AREA AS FUNCTION OF GRAIN SIZE**

Grain Size (axes, mm) x	y	z	Size Factor Decrease	SURFACE AREA INCREASE (X) COMPARED TO 1 CC SOLID VOLUME Cube(y)	Sphere(y)	Plate(xyz)
10	10	0.1	1	1E+00	5E-01	3E+01
2	2	0.02	5	5E+00	3E+00	2E+02
0.2	0.2	0.002	10	5E+01	3E+01	2E+03
0.004	0.004	4E-05	50	3E+03	1E+03	8E+04
4E-05	4E-05	4E-07	100	2E+05	1E+05	8E+06
8E-08	8E-08	8E-10	500	1E+08	7E+07	4E+09
8E-11	8E-11	8E-13	1000	1E+11	7E+10	4E+12

Changes in the conditions of exposure such as wetting, drying, freezing, and thawing affect the pore water ion concentration, and thus the magnitude of the osmotic force. These changes are reflected in the volume change of the solid. The solid particle itself may be disrupted, or, by virtue of expansion, exert disruptive forces on the enclosing matrix, fracturing it (as in the case of an aggregate particle in concrete).

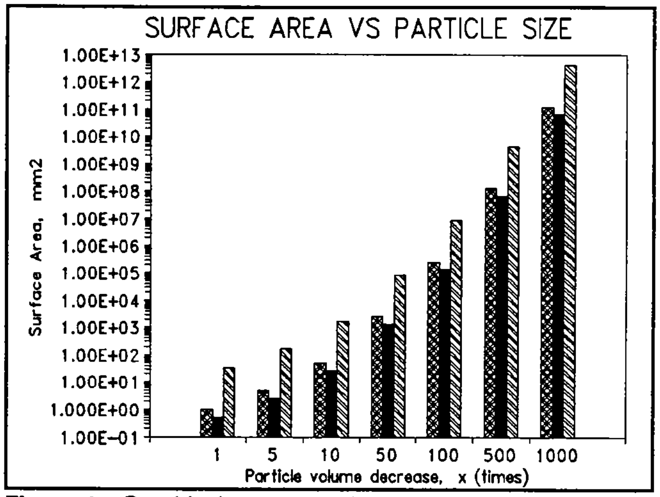

Figure 1: Graphical representation of Table 1.

Not all ions are equally disruptive. The cation with high charge/ionic size ratio, such as Na, behave more aggressively than a larger K ion. The aggressive behavior of the ions is not restricted to creating destructive osmotic forces. Na and K cations are involved in an alkali-silica and alkali-carbonate reaction that has proven destructive to concrete.

The cation in the de-icing salt is thought to be preferentially attracted to the pore surface, and in turn attracts the polar water molecules to itself. Three ways to decrease or eliminate the deleterious effects of salt cations is to either not allow them access to the pores (semi-permeable or impermeable membrane), displace them by other ions, or satisfy their charge and thus minimize their attraction for water molecule. It is this last approach that was chosen for the research described in the following sections.

If some ions are more disruptive than others, then replacing, complexing, or otherwise distracting these ions from the surfaces of pores may ameliorate their behavior. One approach is to seal the surface so that no ion-laden waters may reach the pores. Application of seals (mostly organic) has met with limited success. Another method may be to introduce other ion(s) that will interfere or compete with the aggressive ones. This approach was taken in current research. Some phosphate compounds have been found effective in this function. The research is continuing to isolate other compounds that may also work.

EXPERIMENTAL APPROACH

General durability and response of aggregates to freezing and thawing can be determined by a variety of direct and indirect tests. Freezing and thawing of aggregates is perhaps the **most** direct way of determining their resistance to this climatic factor. Several tests exist for this purpose. The freeze-thaw test that gives the most consistent results, is relatively fast, and best approximates the conditions of freezing and thawing in service has been developed by the senior author, and is now used as a specifying test by the Ontario Ministry of Transportation (MTO, 1989, No. LS-614). The test is performed by first saturating the aggregate in 3% by weight of NaCl solution for 24 hours. The aggregate is then placed in a mason jar of sufficient size with 2 to 3 ml of solution remaining in the jar. The jar is sealed, placed on a side, and frozen for 16 hours at -18°C, then thawed at room temperature for 8 hours. The freeze and thaw cycle is repeated five times, the jar rotated 1/4 turn with each cycle. Loss is determined by back-sieving on the same size sieve. A more severe variation of the test is to place the aggregate on a saturated mat or sponge in a flat container rather than the jar.

Other tests employed in this study were water absorption, water adsorption, specific gravity determination, grain size estimation, magnesium sulphate loss, and petrographic number determination (MTO, 1989, No. LS-609).

<u>Effect of Grain Size:</u>

The grain size was visually categorized as 1 = small (fine), 2 = medium, and 3 = coarse. The results of the freeze-thaw test were plotted against these categories, and are shown in Figure 2. The coarse-grained aggregate showed a uniform low loss; as the grain size decreased, the loss increased. The fine-grained aggregate showed a wide range of losses, indicating that the grain size alone is not a good indicator of freeze-thaw resistance. It does, however, suggest that the fine-grained rocks are more

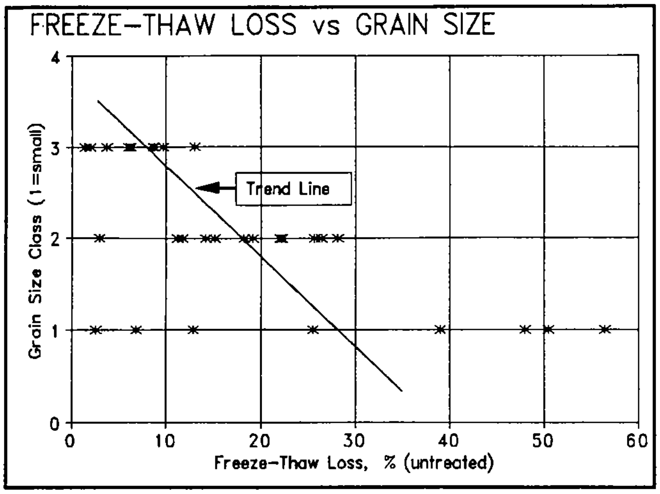

Figure 2 Relationship of grain size on Freeze-Thaw Loss.

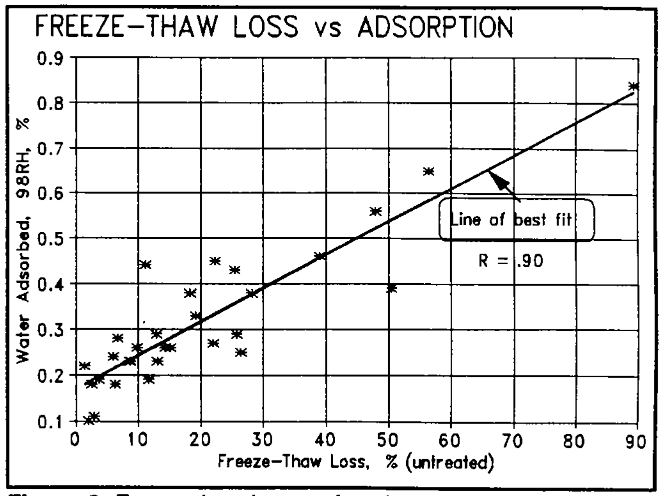

Figure 3 Freeze-thaw loss as function of water adsorption at 98% Relative Humidity, 22°C.

prone to freeze-thaw damage.

The grain size by itself has no effect on durability. The pore size related to grain size does. Fine grain size produces fine pore size, which in turn produces large internal surface area for water and ion sorption. The relationship between water adsorbed at 98% relative humidity and at 23°C and the freeze thaw loss is illustrated in Figure 3. A good correlation is seen, as expected from previous work by the senior author and others (Hudec, 1987). Water is adsorbed on the surfaces of the pore wall; if the pore is small enough, the adsorbed water can completely fill the pore. The pore thus contains water of lower vapor pressure than the water in the surrounding, larger pores, and becomes a center of osmotic in-flow. Water adsorbed at the humidity of the experiment fills pores less than 1 μm in diameter. The correlation suggest that the more small pores the rock contains, the more prone it is to freeze-thaw damage. These pores can be termed "force pores," in that they are probably responsible for the expansive forces that expand and deteriorate the rock.

If the rock is saturated in the de-icing solution, dried, and then exposed to the same humidity conditions, the amount of water adsorbed increases. Figure 4 shows the relationship between water adsorbed on the same sample before and after treatment with 3% NaCl salt solution. It is seen that the salted specimens adsorb approximately 10% more water than the unsalted ones under same conditions of exposure. The increased adsorption implies that either 10% more pores are filled, or pores that are 10% larger in diameter are filled. The effect is to increase the number of "force pores," and therefore the amount of expansion the rock particle experiences. The increased number of 'force pores' leads to greater deterioration. The above discussion is one explanation why the de-icing salts tend to be harmful even though they decrease the amount of water freezing in the rock.

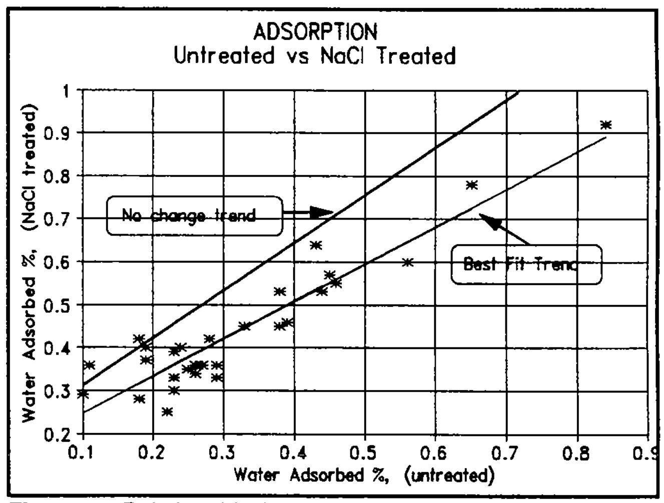

Figure 4 Relationship between water adsorbed before and after treatment with 3% NaCl salt solution.

The effects of grain size, pore size, and water sorption in the samples studied can be summarized in a triangular diagram originally published by Dunn and Hudec (1972), and shown in a slightly revised form in Figure 5. It illustrates the 'critical saturation' effect-- i.e., it separates the rocks that tend to saturate to over 80% of total void space when immersed in water. It further classifies the rocks that critically saturate by either immersion (designated as FROST SENSITIVE), or by high humidity alone (designated as POOR). The latter tend to deteriorate by wetting and drying alone, as well as by freezing and thawing. The diagram shows the importance of water sorption on durability properties of rocks. Salt increases both water adsorption and critical saturation in porous materials, moving the position of the sample toward the POOR area of the diagram.

CHEMICAL TREATMENTS AND RESULTS

Treatment chemicals selected for experimentation had to have potential for beneficiating effect and had to be non-toxic and environmentally acceptable as well as economically viable. The Amine group of chemicals was tried originally and proved to be effective; however, they are noxious chemicals, and being organic, are probably degradable with time. Phosphate group was finally selected as the most promising--inorganic, inexpensive, and benign. Ammonium phosphate $(NH_4)_2$ HPO_4, and various Na, K, and Ca mono- di- and tri- phosphates were tested.

The chemical treatment was applied as follows, and in each case followed by five-cycle freezing and thawing:

A. Pre-treatment of selected dry aggregates.
B. Exposing aggregate to de-icing salt, drying, and then treating it with the chemical.
C. Casting of mortar and concrete samples:
   i. with pre-treated aggregate
   ii. with non-treated aggregate and then treating the concrete or mortar
D. Mixing of chemicals in varying proportions with the de-icing salt, and using the mixture as saturating medium for freeze-thaw tests. The purpose of this procedure was to determine if the normal de-icing salt can be made less aggressive and still retain its de-icing properties.
E. Influence of the following variables:
   i. chemical concentration
   ii. length of treatment
   iii. leaching of treated aggregate by water.

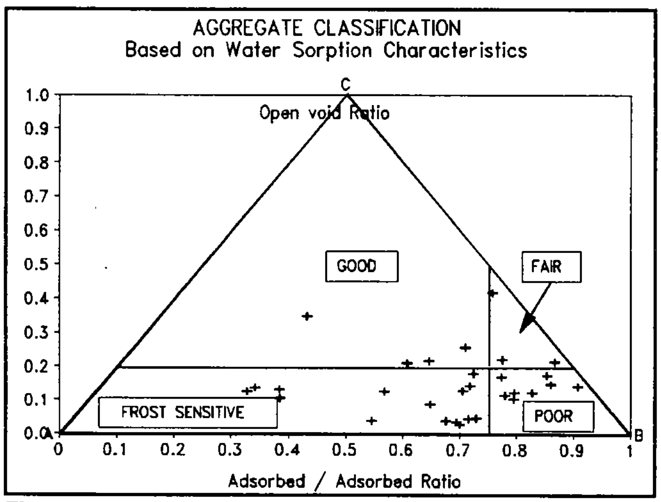

Figure 5 Tri-plot of sorption values related to freeze-thaw resistance.

Only limited results of these experiments are presented in this paper; a more thorough compendium of results will be presented when all the experimental work is completed.

<u>Treatment of Aggregate with Na- and Ca- Phosphate:</u>

Figures 6 and 7 show the effect of treatment of dry aggregate by calcium and sodium phosphate, respectively. Calcium phosphate is relatively insoluble in water and must be acidified to below pH 3 to be brought to solution. It is, however, slightly soluble in NaCl solution. Sodium phosphate (as most sodium compounds) is fully soluble. The treatment in both cases consisted of immersing dry aggregate sample in the phosphate solution for a period of 1 minute. The sample was then allowed to bench dry before being placed in a 3% NaCl solution for 24 hour saturation prior to freeze-thaw testing.

The effectiveness of treatment is compared to freeze-thaw test results on the same untreated aggregates. As the figures show, there is a significant improvement in the resistance to freezing and thawing of the treated aggregates under what can be considered very severe conditions. It is interesting to note that the higher the freeze-thaw loss of the untreated sample, the more effective is the treatment in reducing the loss.

Figure 8 compares the effectiveness of the two phosphate treatment chemicals. Calcium phosphate is, in general, about 20% less effective than sodium phosphate. The acidified calcium phosphate tends to precipitate in the surface environment of the aggregate particle, especially

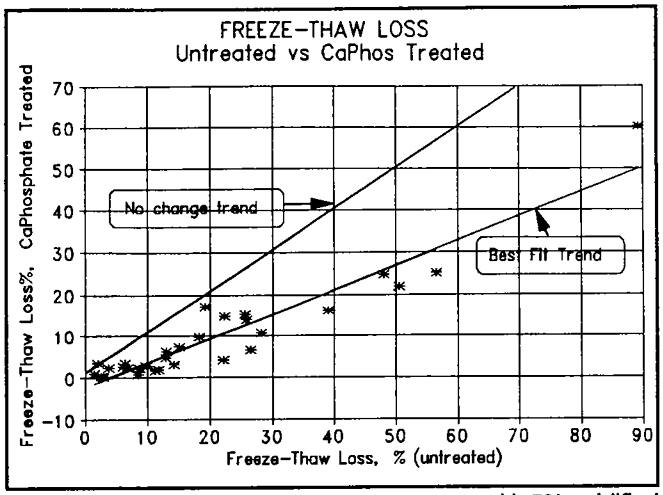

Figure 6. The effect of aggregate treatment with 5% acidified calcium phosphate for 1 minute.

when the aggregate is, or contains, carbonate. The precipitated calcium phosphate may, however, make the calcium treatment more effective in the long run, because the precipitated phosphate will stay with the aggregate; sodium phosphate, because of its high solubility, may be subject to leaching. Although insoluble in water, calcium phosphate is slightly soluble in NaCl solution--the solution that causes most damage to concrete.

Figure 9 illustrates the effect of concentration of sodium phosphate in the freeze-thaw solution on the freeze-thaw loss. The concentrations are expressed as molar ratios--i.e., ratio of number of molecular weights of NaCl to Na_2HPO_4. The molar ratios of 1, 3, 6, and 30 translate to phosphate concentrations in the salt mixture of 71, 49, 29, and 7.5 respectively. It is seen that even small amount of sodium phosphate in salt solution decreases the freeze-thaw loss significantly. Sodium or calcium phosphate could thus be used as a salt admixture, providing de-icing action, but reducing the deleterious effects associated with NaCl salt. Phosphate is a well known corrosion inhibitor; simple tests have shown that even small amounts of phosphate in the salt reduces the corrosion of metals. The downside of using phosphates, especially a soluble phosphate, is the potential eutrophication effect of rivers and lakes. More work needs to be done to investigate the environmental aspects of phosphate treatment.

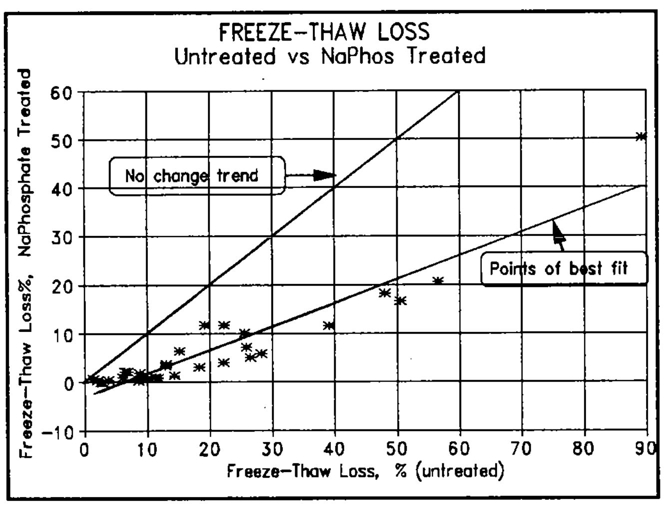

Figure 7 The effect of treatment of aggregate with 5% sodium phosphate for 1 minute.

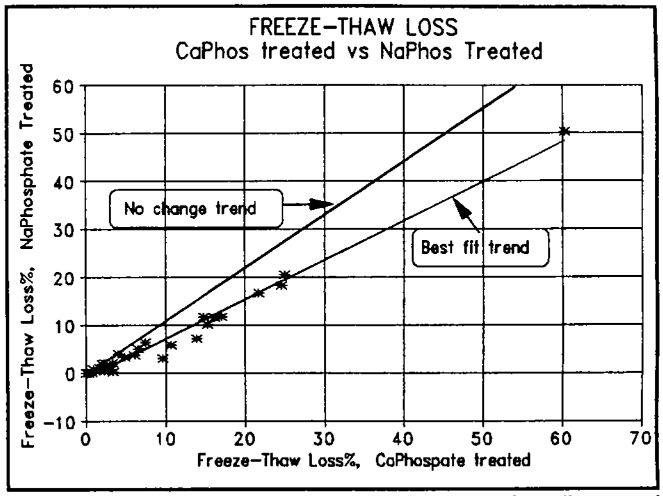

Figure 8 Comparison of the effectiveness of sodium and calcium phosphate treatments.

Ammonium phosphate was also shown to be an effective treatment for aggregates. Because it contains ammonium, the salt is somewhat noxious in large concentrations, but it is probably acceptable in amounts that were found to be effective. Figure 10 shows the effect of treating previously salted aggregate with ammonium phosphate solutions of varying concentrations by five minute immersion. It is seen that the effectiveness of the treatment is a log function of concentration--i.e., very small concentrations have a significant effect, and increasing the concentration of the phosphate results in some increase in benefit, but not proportionate. Concentrations of 1 percent of this chemical would appear to be the optimum in terms of concentration and effect. Although not compared directly to other phosphates, ammonium phosphate would seem to be more effective in lower concentrations

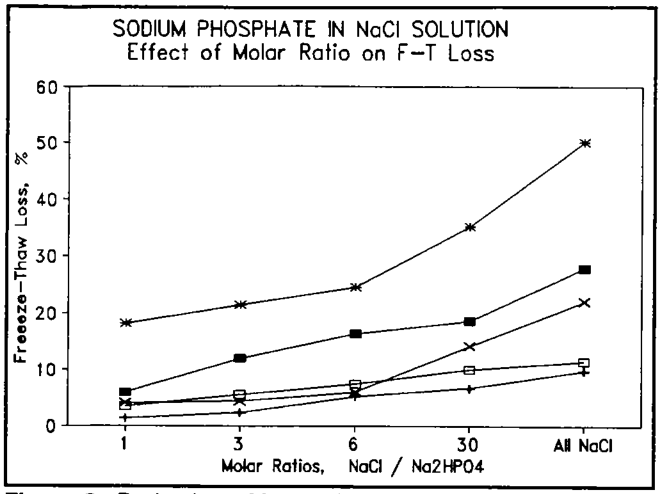

Figure 9: Reduction of freeze-thaw damage by admixtures of Na_2HPO_4 in NaCl.

than either sodium or calcium phosphate.

The effect length of exposure to the treatment chemical on freeze-thaw deterioration is considered next. Figure 11 gives the results for ammonium phosphate.

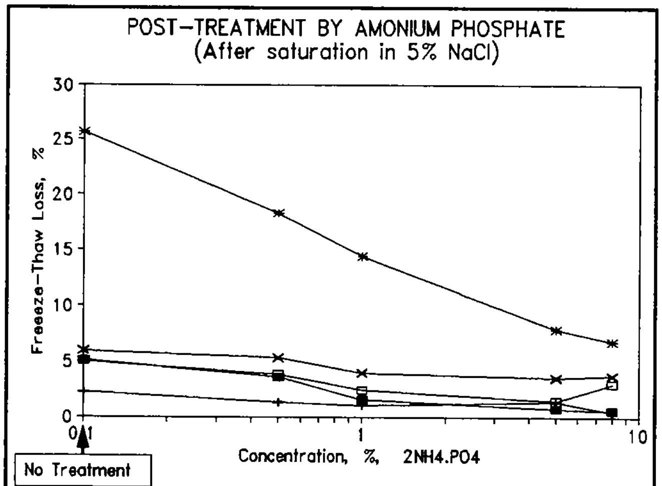

Figure 10: Treatment of NaCl-saturated aggregates with ammonium phosphate and its effect on freezing and thawing loss.

Dry aggregate was exposed to a 3% solution for 1, 10, 100, 600, and 1000 minutes. The relationships are shown to be log-normal, i.e., the longer the treatment, the more improvement there is; however, the amount of improvement is not significant after certain period of exposure. From a practical point of view, the most effective exposure time is between 5 and 10 minutes.

The question of effectiveness of treatment with time was addressed in the next set of experiments. Samples treated in 3% ammonium phosphate solution for 5 minutes were allowed to dry, and then leached in running water for 24 hours. After drying, the 24-hour leaching cycle was repeated two more times. The samples were then subjected to freezing and thawing. The results on five different aggregate samples are given in Figure 12. It is seen that leaching did decrease the resistance to freeze-thaw degradation, but the amount of decrease was marginal. The minimum decrease of effectiveness following leaching suggests that this treatment chemical is effectively bound to the internal surfaces of the aggregate pores, and is able to withstand severe leaching. The laboratory leaching was probably more severe than may be encountered in service.

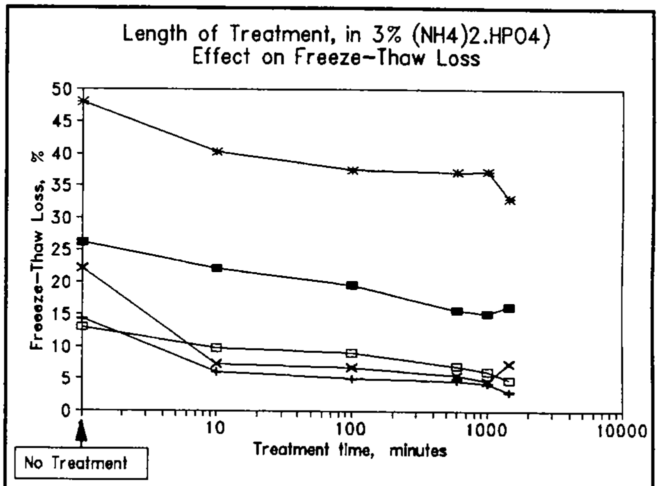

Figure 11: Effect of length of exposure to the treatment chemical on freeze-thaw durability.

DISCUSSION AND CONCLUSIONS

The results presented above show that some chemicals, especially phosphates, are effective in controlling the destructive effect of de-icing salt during freezing and thawing. The effectiveness of phosphates is probably due to the ability of the phosphate anion to bind to and satisfy the charge of the Na^+ ion. The phosphate anion thus reduces the clustering of polar ater molecules around the Na^+ ion, decreasing the osmotic pressure difference between the water in very small pores and the water in larger pores or the outside of the aggregate.

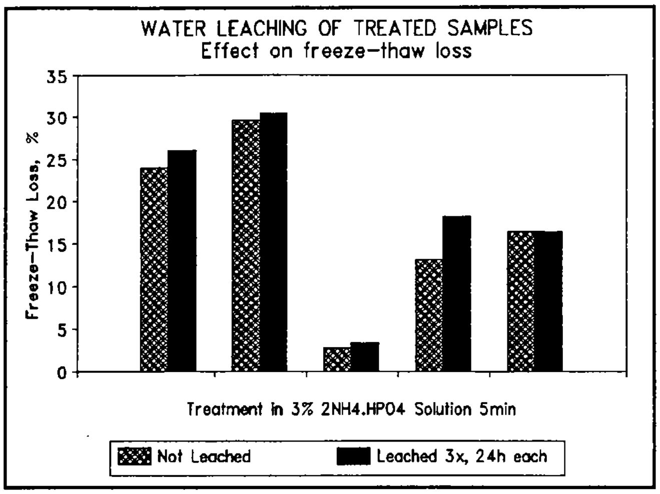

Figure 12: Effect of leaching of treated aggregate on the freeze-thaw durability.

The treatment by surface-active chemicals applies not only to aggregate, but also to any porous building material. Surface treatment of mortar and concrete was equally successful in suppressing the freeze-thaw damage from de-icing salts. Some initial tests suggested that incorporating small amounts of some of the treatment chemicals in the mortar and concrete mixtures may also be an effective way in reducing the de-icing salt damage. More work is continuing along these lines.

The surface-active chemicals have a potential in being used in three different ways:

1. Topical or surface application to aggregate prior to its use in concrete.
2. Surface treatment of concrete or mortar.
3. Chemical admixture (in mixing water) during concrete batching.

It should also be mentioned that, not surprisingly, the phosphates were found mildly effective in reducing alkali reactivity of concrete and mortar under conditions 1 and 3 above. Alkali silica and silicate reaction takes place when free alkalis Na and K abound in hardened concrete that contains reactive aggregate. The alkalis are derived from the cement and from the de-icing salts. The phosphate, because of its affinity for any free cation, binds with the alkalis and reduces the alkali reaction. The ability of phosphate to reduce both the freeze-thaw damage and alkali reactivity is an added incentive to investigate further the use of surface-active chemicals to improve the resistance of mortar and concrete.

ACKNOWLEDGMENT

The work described above was supported by an operating grant from National Science and Engineering Research Council of Canada (NSERC) to the senior author.

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Highway Bridge Failure Due to Foundation Scour and Instability

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INTRODUCTION

Highway bridges most often fail as a result of flooding, with the most common cause of damage being scour of abutment or pier foundations (Harrison 1991). Typically, erosion-related failures are attributed simplistically to "scour." The actual process of failure may be a complex interaction among several mechanisms, but soil removal by tractive current forces is considered the dominant mechanism of erosion (Breusers, Nicollet, and Shen 1977). Moreover, slope instability at abutments often is caused by a combination of mechanisms involving scour of the toe of the slope, localized collapses in the slope face due to seepage outflow, and subsequent mass instability in the over-steepened slope. These other mechanisms and their interactions with scour forces may be the primary causes of bridge failures at some sites.

The great majority of the highway bridges in the United States are small, short-span structures founded at least partially in or on alluvial soils. Consequently, the erosion mechanisms most significant to removal of alluvial soils should be of most importance to stability of highway bridges. Those mechanisms include the widely recognized processes of tractive force scour and mass slumping (or sloughing). However, another general class of mechanism may be very important to erosion of alluvium and consequent instability of bridge components. This class of mechanisms is driven by seepage.

The time-variant character of this seepage-induced erosion is important in explaining bed erosion around bridge piers. The variation in seepage rate and direction that occurs in natural streams at bridges also is very important in explaining mechanisms of bridge instability, including scour at piers and slope instability at abutments.

Many researchers have recognized the impact of vortex systems and flow accelerations that develop in the vicinity of highway embankments and bridge foundations; however, few investigators have examined the effects of groundwater seepage on erosion in the vicinity of bridge crossings. Some researchers have shown that seepage may influence the development of scour holes, and others have shown that slope stability and sediment transport are significantly influenced by groundwater seepage. A comprehensive review of seepage-induced erosion processes and conditions that may occur in the vicinity of bridge sites shows that seepage plays an important role in scour at bridge sites and must be considered in designing erosion protection. This paper describes the role of seepage in alluvial erosion processes and describes the effects of seepage on streambank and streambed erosion at bridge sites.

SCOUR AT BRIDGE SITES

Erosion at bridge openings can be classified as general scour, constriction scour, or local scour. General scour is the erosion associated with the lowering of the entire stream profile or with the lateral migration of the stream. Constriction scour is the erosion associated with the reduction of flow area caused by highway embankments that encroach on the floodplains, by large bridge piers or abutments, by bridge superstructure components (if the bridge is overtopped), and by debris blockage. Local scour is caused by local flow phenomena and is described in detail below.

Local scour at bridge piers and abutments is a result of 1) locally high boundary stresses caused by secondary currents and associated vortex systems; 2) a diversion of sediment away from the region of high boundary stress caused by secondary currents; and 3) locally high hydraulic gradients in the surrounding streambed caused by abrupt changes in water surface elevation that are especially severe near flow separation points. Most researchers who have studied the process of local scour have concentrated on the boundary stresses associated with the horseshoe vortex that tends to form on the upstream nose of blunt piers (Breusers, Nicollet, and Shen 1977).

Hjorth (1975) showed that the downflow that occurs at the upstream nose of the pier is composed of water diverted from the upper portion of flow just below the water surface; this water may contain suspended sediments but is essentially void of bed load sediments. As this water impinges on the bed on the upstream side of the pier, it forces water from the bottom layers of flow that contains bedload sediments away from the pier area (Figure 1). Hypothetically, if the boundary stress around the pier is equal to the boundary stress of the surrounding streambed, net erosion will still occur because any particle that is removed from the vortex zone will not be replaced by inflowing sediment.

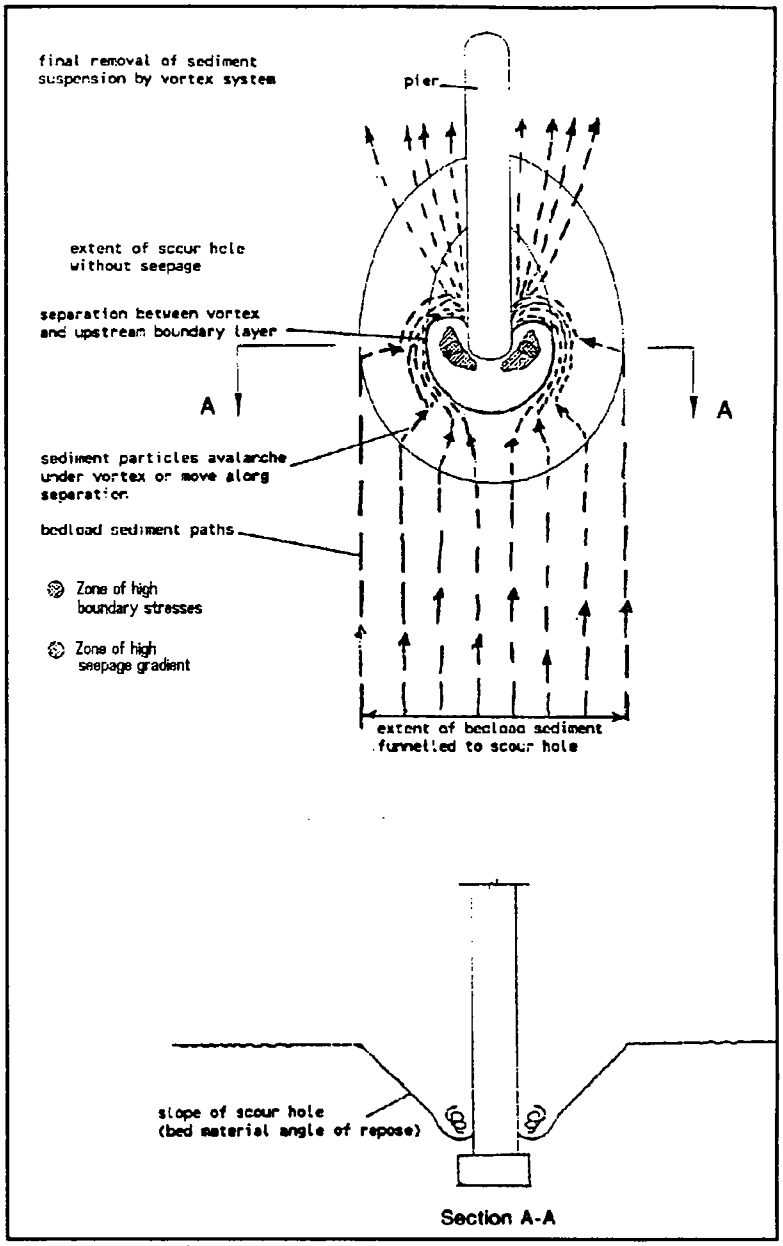

Figure 1: Local Scour at piers.

The authors of this paper have conducted several experiments to determine the paths of bed load sediments in the vicinity of rectangular flow obstructions. Several 5 mm plastic spheres with specific gravity equal to 1.1 were placed on the bed upstream from a 7.5 cm-wide model rectangular pier. The path of the particles was monitored closely during several tests that used varying flow rates. In every situation, the spheres stopped before entering the region of high boundary stress and were diverted around the region of the horseshoe vortex. A substantial scour hole was required before the plastic spheres could enter the region of the horseshoe vortex by avalanching down the slopes of the scour hole. The authors performed a second series of experiments in which filaments of string 1cm in length were attached to small anchor pins which were driven into the sand bed around a model bridge pier in a 1 cm by 1 cm grid pattern. Water moving near the streambed tended to align the filaments with the mean velocity direction of the flow. The configuration of the filaments indicated that water and the bed load sediment conveyed in the lower layers of flow are diverted around the location of the horseshor vortex. This experiment supports the results of the plastic sphere experiments and the hypothesis of Hjorth (1975).

In practice, general, constriction, and local scour are considered independent and are superimposed in estimates to compute total scour; however, this simple superposition probably does not adequately represent the actual situation. Erosion at the pier may cause instability of the main channel that in turn causes lateral instability. Likewise, lateral migration of the stream may change the flow field in the vicinity of piers and drastically change the amount of local scour at piers or abutments. Erosion at bridges is a complex combination of erosion processes, to say the least. Although there is a great deal of interaction among the different classes of scour, the effect of seepage on each scour process can be considered independently.

HYDRAULIC GRADIENTS IN ALLUVIUM AT BRIDGE SITES

Hydraulic gradients in alluvial channels in the vicinity of bridge sites are a result of 1) differences in hydraulic head between the stream and the surrounding ground water; 2) differences in water surface elevations between the upstream and downstream reaches of the bridge approach embankments caused by backwater; and 3) local differences in bed pressure caused by local accelerations of flow at piers and abutments.

General Inflow and Outflow Seepage

General seepage into and out of the stream occurs as a result of differences in total head, including groundwater elevation and porewater pressures in the alluvium and the water surface elevation of the stream (Figure 2a). Groundwater hydraulic gradients may be particularly large during storm events. As floodwaters rise in the stream, lag in porewater pressure behind stream pressure causes flow into alluvial streambanks and beds, resulting in groundwater recharge. As floodwaters recede, the hydraulic gradients may be reversed, causing seepage into the stream from the streambed and banks.

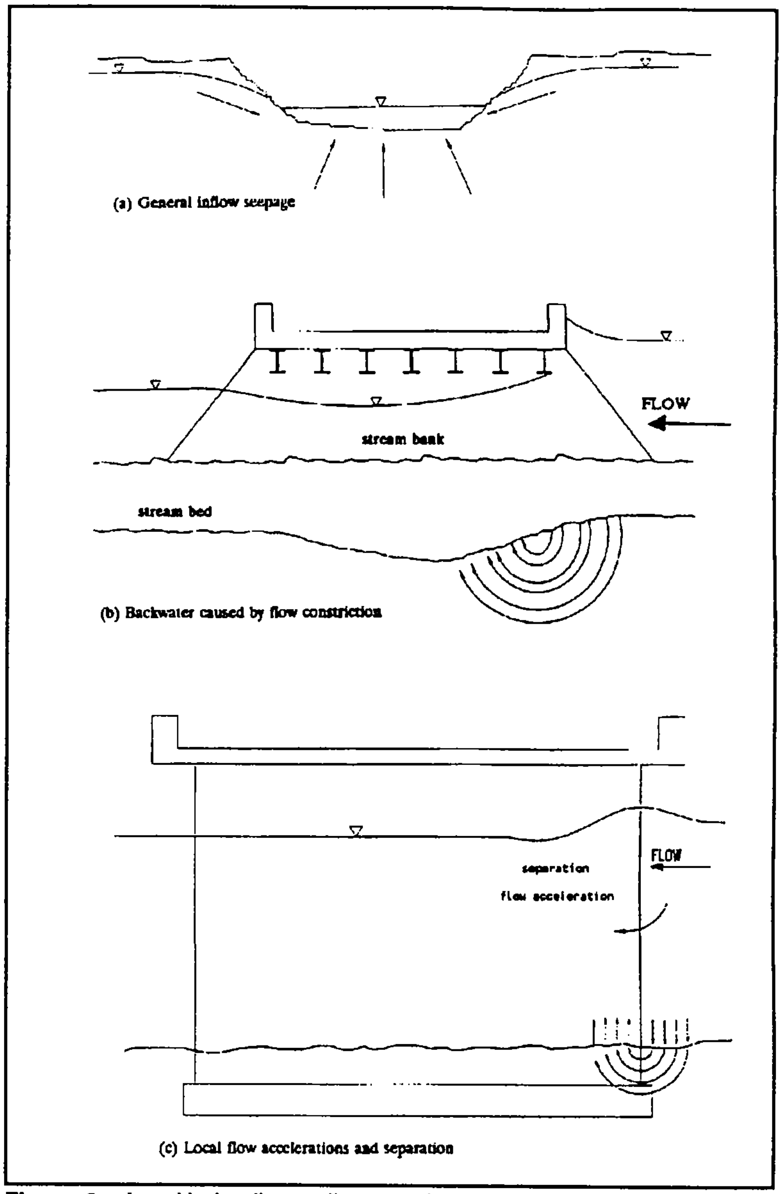

Figure 2-a,b,c: Hydraulic gradients and seepage at bridge crossings.

The location of inflow-outflow is affected by layering in alluvial soils. In layered alluvium composed of beds containing sand, silt, and clay, hydraulic conductivities in the horizontal direction typically are much higher than overall hydraulic conductivity in the vertical direction. Flow thus occurs principally horizontally into and out of sandy layers.

Backwater

Seepage into a bridge embankment and the surrounding bank alluvium is caused by hydraulic gradients created by backwater effects (Figure 2b). The magnitude of the gradient is a function of the degree of flow constriction. Highway embankments and abutments built across floodplains often constrict floodplain flow significantly during flood events. Undersized bridge openings may cause extensive backwater and produce a significant difference in water surface elevation between the upstream and downstream sides of a bridge. Backwater effects are most likely to be significant when 1) the bridge opening is severely undersized for the flow event; 2) debris has partially clogged the bridge opening; 3) the upstream water surface has encountered the bottom bridge beam, causing pressure flow; or 4) the downstream water surface has dropped suddenly because of tidal recession.

Local Flow Accelerations

In the vicinity of bridge abutments and piers, local flow accelerations may be relatively large as flow changes direction in relatively short distances. In most cases, flow diverges from piers and abutments to create separation points. At these locations, the water surface elevation dips drastically over short distances. The dip is especially pronounced in the flow around the upstream corners of rectangular-nosed.

The pressure at the stagnation point in front of the pier is greater than alongside the pier where the pressure corresponds to static head; the difference in pressure corresponds to the full velocity head. The pressure difference induces a flow through the bed that can cause quick conditions where seepage emerges (Figure 2c). Posey (1974) estimated that the competence of the velocity beside the pier is about equal to twice the average competence. The turbulent vortices that occur at sides of bridge piers combine with the effects of seepage outflow to remove particles around the pier (Figure 3). Both the vortices and the seepage forces pulse and fluctuate so that the focus of erosion shifts around the pier with time.

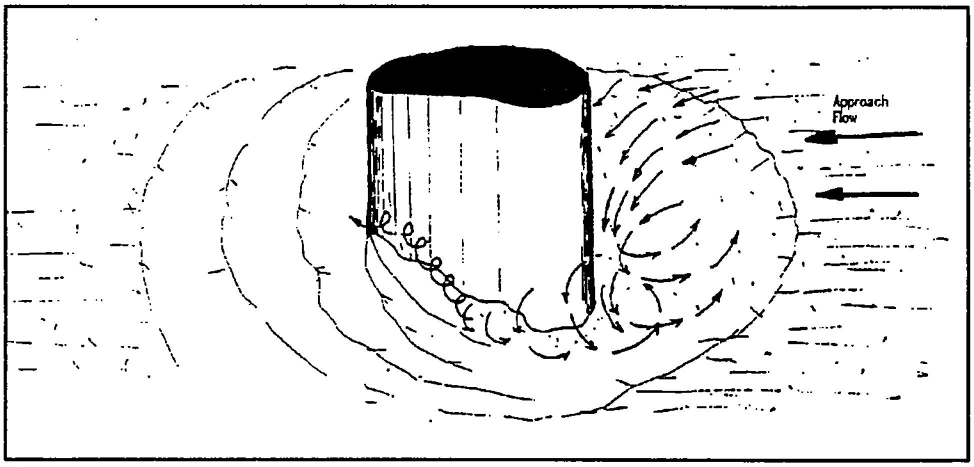

Figure 3: Vortices and scour around a pier.

In a comprehensive study of scour around piers, Hjorth (1975) tested scour around circular, rectangular, and sharp-nosed piers, in a flume. During these experiments, Hjorth measured pressure differences in the water around the pier and in the porewater between the bed particles; he concluded that "...consideration of the resulting seepage forces shows that these should absolutely be included in the stability analysis of the foundation." The results of these tests confirmed that outflow seepage alters bed forms significantly, but more importantly, they showed that pressure differences and horseshoe vortices both are unstable and fluctuate periodically. Flow and pressure measurements showed that the diving motion in the vortex coincides with positive pressures at the bed surface (at the pier face) equal to the full velocity head. This head would create seepage into the bed. Beside piers, pressures decreased by two full velocity heads from the pressures measured remote from the pier (Figure 4a). Outflow seepage would occur in response to such pressure differences beside and behind piers. Measurements of pressures in porewater inside the bed at a distance of one-half the pier diameter from the pier side showed differences of 0.5 to 0.6 times the average velocity head. Hjorth's results confirmed findings by Muller et al. (1971) that a lift force of 0.7 to 1.0 times the buoyant weight of a particle would be sufficient to entrain that particle into rolling or sliding bed motion.

In the highly turbulent wake zone close to the pier, outflow gradients would be much higher and undoubtedly would approach bed liquefaction values. The combined seepage outflow effects and vortex forces are primary factors in causing scour. These effects were very high at the front corners of rectangular piers where vortices were concentrated and pressure differences were equal to three full velocity heads between a point just in front of the corner and a point just around the corner, in Hjorth's tests (Figure 4b).

Ettema (1980) also focused attention on the fluctuating vortices and pressure differences around piers. His test results showed that the vortices and turbulent eddies around a pier can lower the local pressure and cause particles to be ejected from the bed by porewater

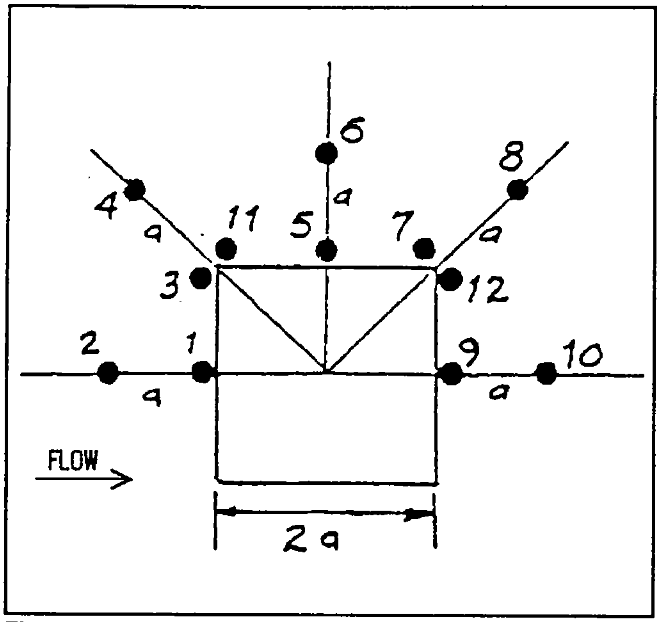

Figure 4a: Location of pressure taps around pier.

pressure, even apparently sheltered particles. Ettema's tests confirmed the findings of Melville (1975) that the horseshoe vortex at the head of a pier initially is small in cross-section and weak, as is the downflow at the pier front; after the scour hole is initiated, the vortex grows rapidly in size and strength, as does the downflow; eventually, the scour hole contains the complete vortex. No adequate analytical descriptions of this process were developed. Ettema did recognize the temporal nature of the development of the scour hole; he described an initial phase of hole development, an enlargement or eroding phase, and a final equilibrium phase. He postulated that scour during many flood events does not reach the equilibrium phase and offered that consideration as an explanation for the behavior of some bridges that have failed only after a number of similar flood events. Change in discharge rate occurs much more rapidly in many streams than does the development of scour features. Scour developed in Ettema's tests, as in Hjorth's tests, first beside and at the rear edges of piers, and then progressed to the front corners or front face of the pier. Ettema reported, "Simultaneously to the scour initiated by a shear stress excess caused by the contraction of stream lines near the sides of the pier, the sediment immediately behind a pier is entrained by the action of the cast-off vortices and eddies in its wake. These reduce the hydrostatic pressure behind the pier and entrain bed particles largely by ejecting them from the surface of the bed." (Ettema pg. 185) However, Ettema did not consider the full significance of the time-variant nature of pier scour: that time variation is intimately related to the seepage conditions around a pier.

SEEPAGE-RELATED EROSION PROCESSES

Instability at bridges can consist of mass slumping of embankment and streambank slopes with consequent failure of abutments. In certain circumstances, instability consists of concentrated erosion of portions of streambanks with consequent localized failures of intermediate-sized soil masses. At the extreme range of size considerations, instability can consist of removal of an individual soil or rock particle near a pier, on the streambed, or on the face of the streambank. All of these modes of instability are influenced significantly by seepage, from stream into bed and bank or from the soil

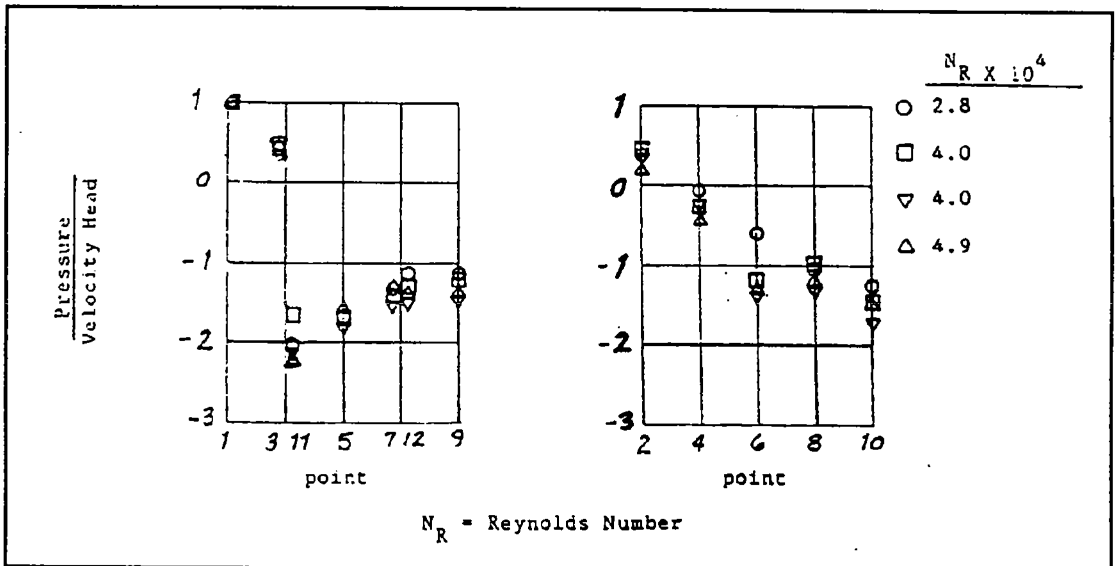

Figure 4b: Pressure differences around a pier (after Hjorth).

into the stream.

Slope Stability

Seepage flow out of the banks and bed of a stream occurs when the gradient in total head in bed or bank porewater is directed toward the stream. Porewater pressures in the soil generally exceed pressure in the stream except during floods; these excess porewater pressures and the particle drag forces developed by the seeping water long have been recognized by geotechnical engineers as very important factors in destabilizing bank slopes (Taylor 1948). However, most geotechnical slope stability studies have not included the influence of flowing surface water on the slope. Many investigators have recognized the effect of scour on bank stability (e.g., Thorne et al. 1988), but few have given any attention to bank erosion by internal seepage, or piping/sapping (Hagerty 1991a).

Likewise, many geomorphologists have observed that seepage forces change the erodibility of soils (DeVries 1976) and that seepage erosion is widespread in many natural landscapes (Dunne 1980, 1988; Higgins 1984; Higgins and Coates 1990). These investigators uniformly have emphasized the influence of outward flowing groundwater in reducing effective soil particle weight and have inferred that such weight-reduction effects would facilitate sediment entrainment and the development of channels. Conversely, seepage in the streambanks and beds has not been examined significantly by many geomorphologists.

Taylor (1948) showed that the slope angle (angle of repose) in cohesionless soil would be reduced by seepage effects; his analysis showed that a change in inflow seepage (seepage into the bed) to outflow seepage (seepage out of the bed) could reduce the stable slope angle by 50 percent or more.

Piping and Sapping

When recharge occurs, inflow seepage effects increase bank stability. As flood waters recede and after the stream returns to its low-flow channel, outflow seepage can initiate grain-by-grain removal. In most alluvial streams, recharge and subsequent outflow are concentrated in the more pervious (sandy) zones between relatively tight fine-grained (silt or clay) zones. When the outflow removes grains from the faces of pervious layers, overlying less pervious layers may be undermined. Removal of support below the fine-grained layers causes those layers to deflect and distort; the most common result of such undermining is the formation of cracks in the top plane of the undermined layer, where the soil is unable to support the tensile stresses created by deflection. The mass in front of such a crack may shear from the bank face if undermining continues and the crack propagates, or it may topple if the undermining induces a tilting motion in the mass defined by the incipient crack. Figure 5 shows typical modes of bank collapse in layered alluvial banks.

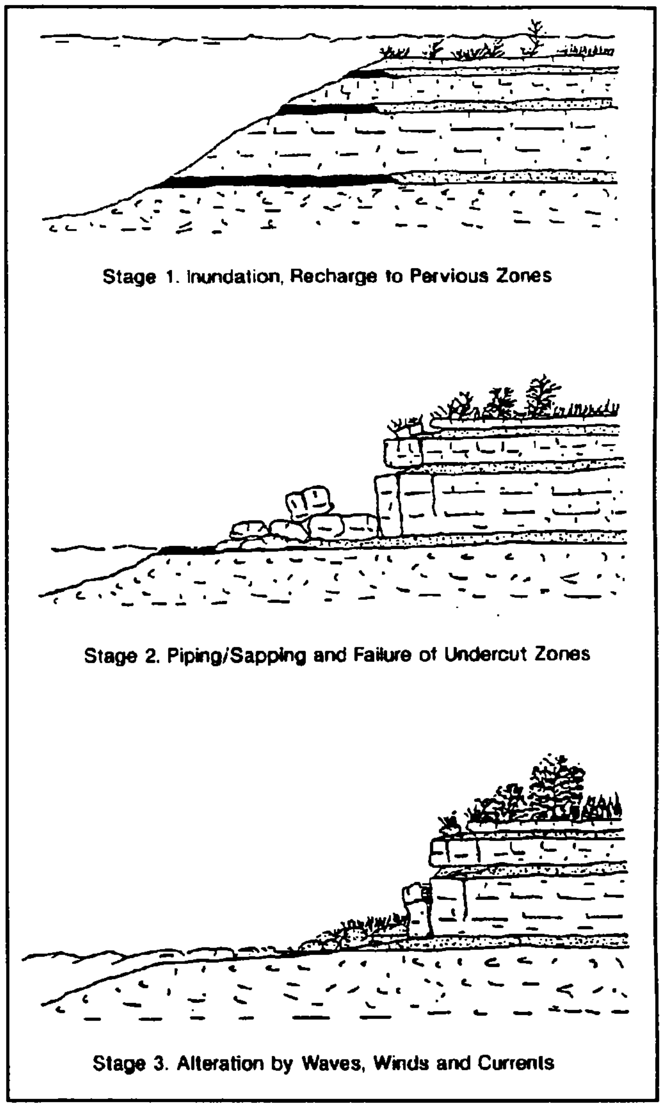

Figure 5: Bank erosion by piping/sapping.

Figure 6a: SEEPAGE OUTFLOW EFFECTS; Soil-water flow out of bank (Coin for scale is a U.S. dime).

Figure 6b: SEEPAGE OUTFLOW EFFECTS; Collapse caused by seepage outflow.

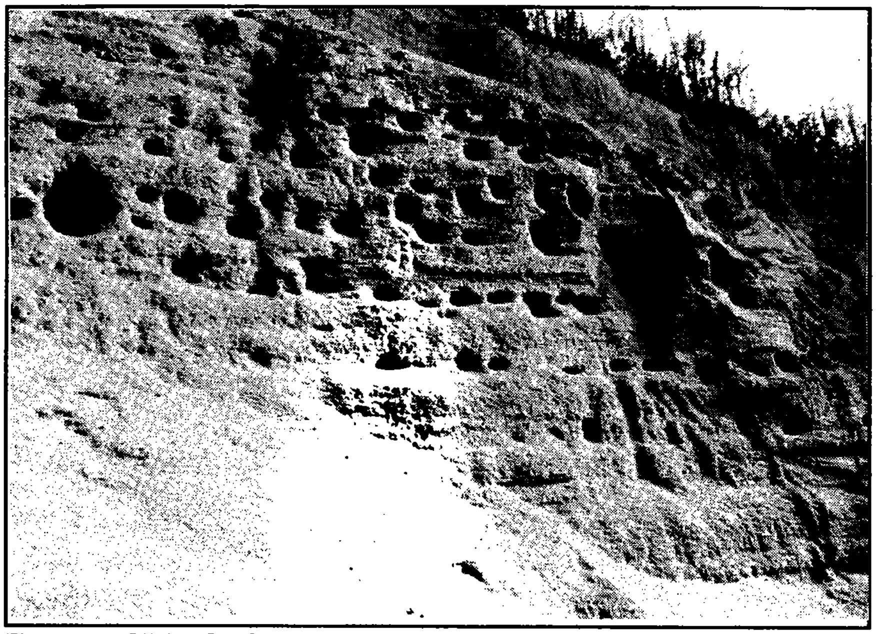
Figure 6c: SEEPAGE OUTFLOW EFFECTS; cavities caused by seepage outflow.

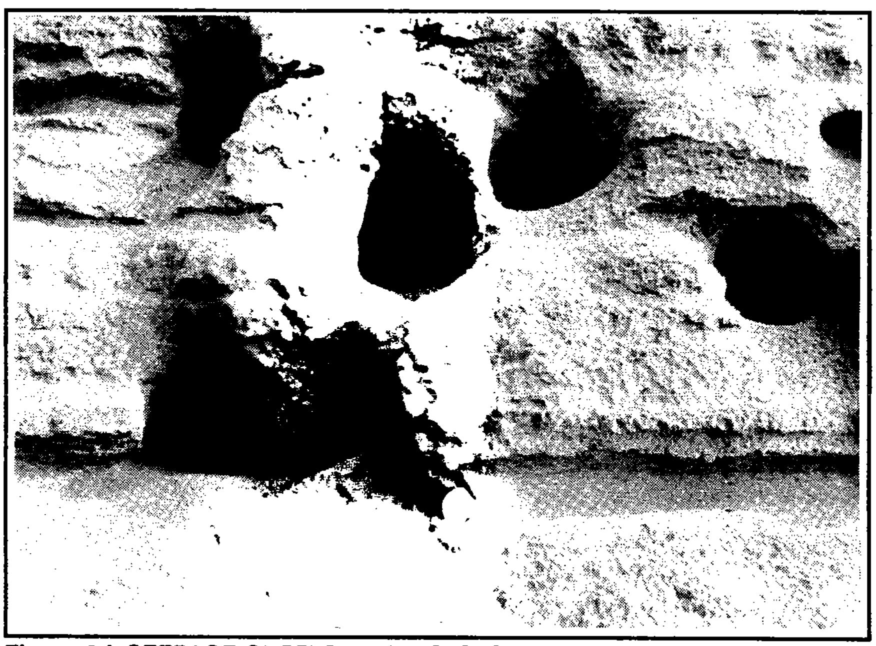
Figure 6d: SEEPAGE OUTFLOW EFFECTS; Close view of cavities (coin for scale is U.S. dime).

The type of failure described in the previous paragraph has been termed a "piping" or "sapping" failure because emerging seepage excavates or "saps" nearly circular cavities in the face of pervious layers, and the cavities tend to elongate into the bank to form pipe-like openings. Figure 6 shows typical cavities of this sort, and Figure 7 shows typical slabs and blocks detached as a result of piping/sapping action. The factors that control this mechanism of failure include the arrangement of soils in layers, as well as the strength, permeability, thickness, and inclination of the bank layers (Springer et al. 1985). The most important hydrological factors are the maximum stage of a flood event and the duration of inundation, because these factors directly control the amount of water that is stored in any given bank layer (and that later emerges to cause seepage erosion and undermining) (Ullrich et al. 1986).

The extent and severity of bank erosion by piping and sapping gradually is being recognized (Hagerty et al. 1986; Hagerty and Hamel 1989; Hagerty 1991a) and is likely to be accorded much more importance in the future because methods for identification and diagnosis of the mechanism have been developed (Hagerty 1991b).

Combined Scour/Seepage Effects on Particles

Watters and Rao (1971) conducted experiments on spherical particles to determine the influence of inflow seepage on drag and lift of an individual particle located in the top layer of a soil matrix and resting on the top layer of the same matrix. They described both direct and indirect effects of seepage on individual particles. Direct effects are dominant when a large portion of the total flow is seepage; the seepage flow produces significant drag and lift on the particle to influence its stability. Indirect effects occur as a result of changes in the channel bed and particle boundary layer due to seepage. Indirect effects include changes in the separation points on fluid particles and changes in the width of the channel boundary sublayer.

Watters and Rao (1971) showed in their analysis that the main effect of outflow seepage from the soil was indirect. Flow into the channel caused a significant increase in the channel sublayer thickness that reduced the drag on fluid particles located in the top of the soil matrix. Seepage into the soil layer caused a significant decrease in the sublayer thickness and an increase in particle drag. These investigators concluded: 1) seepage modifies the sublayer thickness of the channel and the wake pattern behind sediment particles; 2) seepage into the open channel decreases drag regardless of the position of the sediment particle; 3) seepage into the channel increases the lift on a sediment particle located in a plane bed, and the opposite effect was found for seepage into the bed; 4) because drag is dominant in causing particle motion, flow into the stream increases particle stability and flow into the channel bed decreases particle stability; and 5) the influence of seepage is dependent on the boundary Reynolds number.

According to Martin (1971), seepage into the stream does not affect incipient motion of bed particles measurably because the seepage force is lost once a sediment particle rocks. Martin (1971) also concluded that seepage into the bed may either enhance or hinder incipient motion, depending on the relative magnitude of the boundary shear stress and the seepage force.

Burgi and Karaki (1971) conducted experiments to determine the effects of seepage on channel bank stability. They found that erosion of a bank composed of non-cohesive sediment particles was influenced primarily by seepage effects in the bank material when stream flow velocity was less than 1 ft/s, but that erosion at higher channel velocities was dominated by tractive force effects.

Harrison and Clayton (1970) found in flume tests that the slope angle of the downstream limb of bed dunes changed 19 degrees when bed seepage was changed from inflow to outflow seepage.

The research reported by all of these authors indicates that the major effects of seepage acting in conjunction with scour are 1) to directly force particles from the bed if a large exit hydraulic gradient is created in the bed; 2) to indirectly change the characteristics of the channel sublayer and the particle boundary layer; and 3) to change the stable slope angle on bed forms.

Influence of Seepage on Sediment Transport

Hydraulic engineers have studied the mechanics of erosion in alluvial streams for many years, but until fairly recently little experimental or theoretical work was done on seepage effects. In a comprehensive study of flow in alluvial channels, Simons and Richardson (1966) paid particular attention to the relations between stream power/sediment transport and bed form characteristics (ripples, ripples on dunes, dunes, chutes, pools, etc.); they

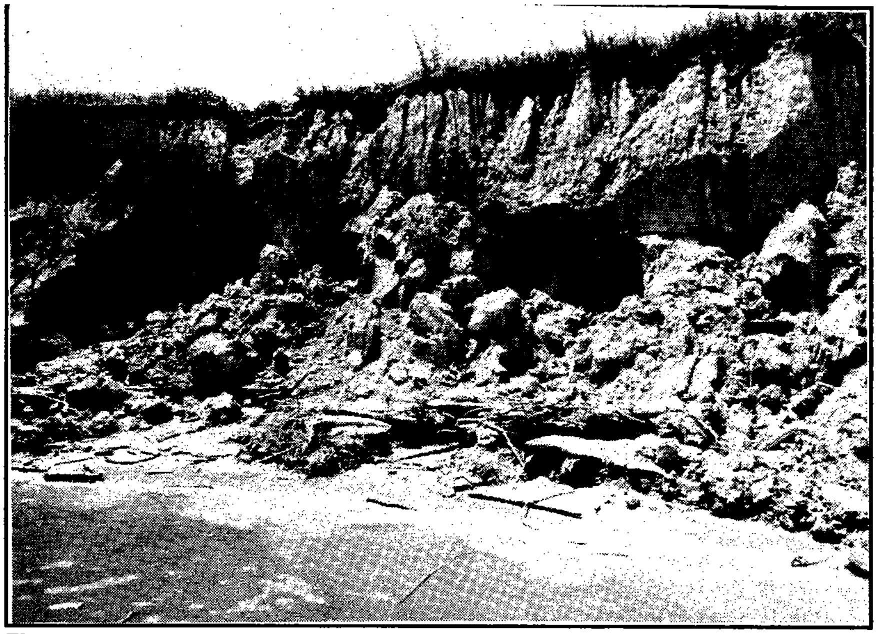

Figure 7a:

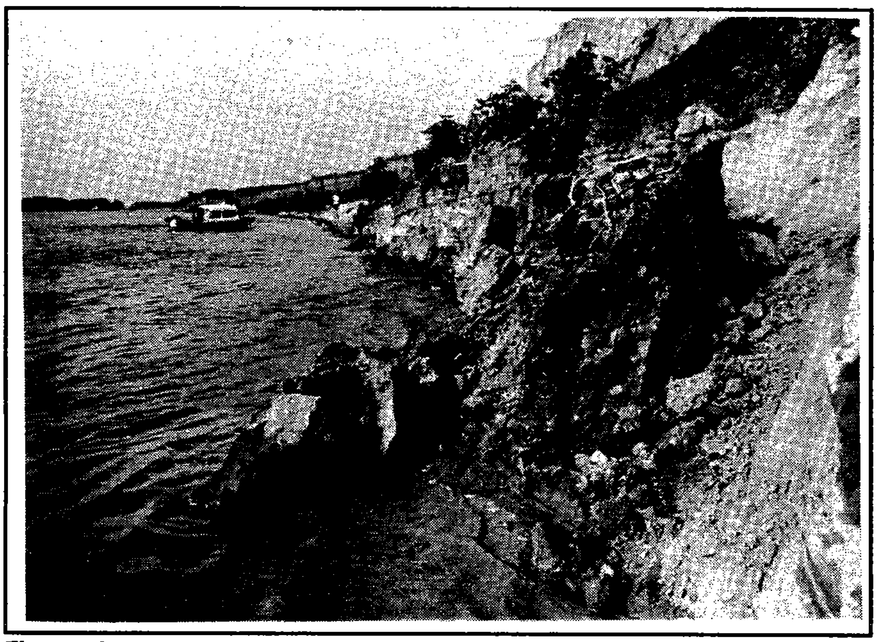

Figure 7b:

Figure 7: Blocks and slabs fallen from bank undergoing seepage outflow.

Figure 7c:

Figure 7d:

Figure 7 (cont.): Blocks and slabs fallen from bank undergoing seepage outflow.

stated that seepage forces could change the bed forms and consequently the resistance to flow. Although no experimental evidence or analytical results had been obtained to test these relations, Simons and Richardson concluded that seepage from a stream into its bed would tend to stabilize the bed while outflow seepage would tend to destabilize the bed, increase sediment transport, and change bed form.

Investigators subsequently attempted to verify the speculations developed by Simons and Richardson. Harrison (1968) and Harrison and Clayton (1970) did not see significant increases in sediment transport or scour in laboratory experiments when seepage varied from inflow (into the bed) to outflow, although they had found evidence in field studies that outflow seepage greatly increased stream transport competence (compared to conditions of recharge seepage into the bed).

In an attempt to clarify the influence of seepage outflow on bed scour, Richardson, Abt, and Richardson (1985) conducted a series of experiments in a laboratory flume. They concluded that seepage into a stream increases localized mean channel velocity, energy slope, and stream power. Seepage outflow had little effect on sediment transport and did not enhance channel scour, according to these investigators, but it did affect channel bed forms.

Although most research by hydraulic engineers has indicated that outflow seepage can change bed forms significantly and affect sediment transport rates, these investigators, in the main, have not concluded that outflow seepage is as effective in increasing erosion as postulated by many geologists and geomorphologists.

In an effort to resolve this conflict, Howard and McLane (1988) reviewed both laboratory and field investigations and concluded that prior slope stability studies had neglected surface flow forces, whereas the channel flow studies had used gradients that were well below the limit of bed stability corresponding to liquefaction. They pointed out that in eroding stream slopes there is a sequence of conditions ranging from wet mass wasting at the heads of erosion channels, through a zone of seepage-induced slurry flow, to transport in channels unaffected by seepage. In a series of laboratory experiments, they demonstrated these conditions. Howard and McLane found that these zones of differing erosional activity move, in time; erosion thus involves mass wastage, piping or "tunnel scour," and seepage-induced transport. The relative importance of each of these mechanisms varies with time at any given location.

SEEPAGE INFLUENCE ON SCOUR AT BRIDGES OPENINGS

General seepage, which occurs in the entire bed and stream channel, backwater-induced seepage, which causes seepage into the upstream bed and banks and out of the downstream bed and banks, and local seepage, which is associated with local flow accelerations, may affect general, constriction, and local scour in different ways. The transient influence of seepage on scour may also be important. Abutment stability may be particularly susceptible to seepage gradients.

Effects of Seepage on General Scour

Seepage from the bed into the stream has been shown by Richardson et al. (1985) to increase effectively the bed elevation in non-cohesive bed materials. If the seepage rate is significant, bed channel characteristics and consequent velocity could be altered appreciably. This change in turn would alter the friction slope and cause general changes in the bed profile and could affect meander tendencies.

Direct seepage effects on the bed of a stream incised in layered alluvium are likely to be small. Vertical seepage is hindered by the low vertical hydraulic conductivity associated with horizontally layered alluvium. In contrast, layered streambanks or subaqueous slopes are very susceptible to seepage erosion. Changes in the streambanks often cause changes in the local flow and cross-section flow characteristics. As a consequence, the entire streambed may become unstable.

Effects of Seepage on Constriction Scour

Constriction scour is the result of increased capacity to convey bed material through the bridge opening because of increased velocities resulting from the reduction in channel cross-sectional flow area. Changes in general seepage in the vicinity of the bridge could affect the amount of sediment transported to the bridge site. This effect within the bridge opening may be similar to behavior in the approach to the bridge and downstream of the bridge. The direct effect of bed-to-stream (effluent) seepage on constriction scour is likely to be small in non-cohesive sediments. Likewise, the effect of general bed effluent seepage on constriction scour in

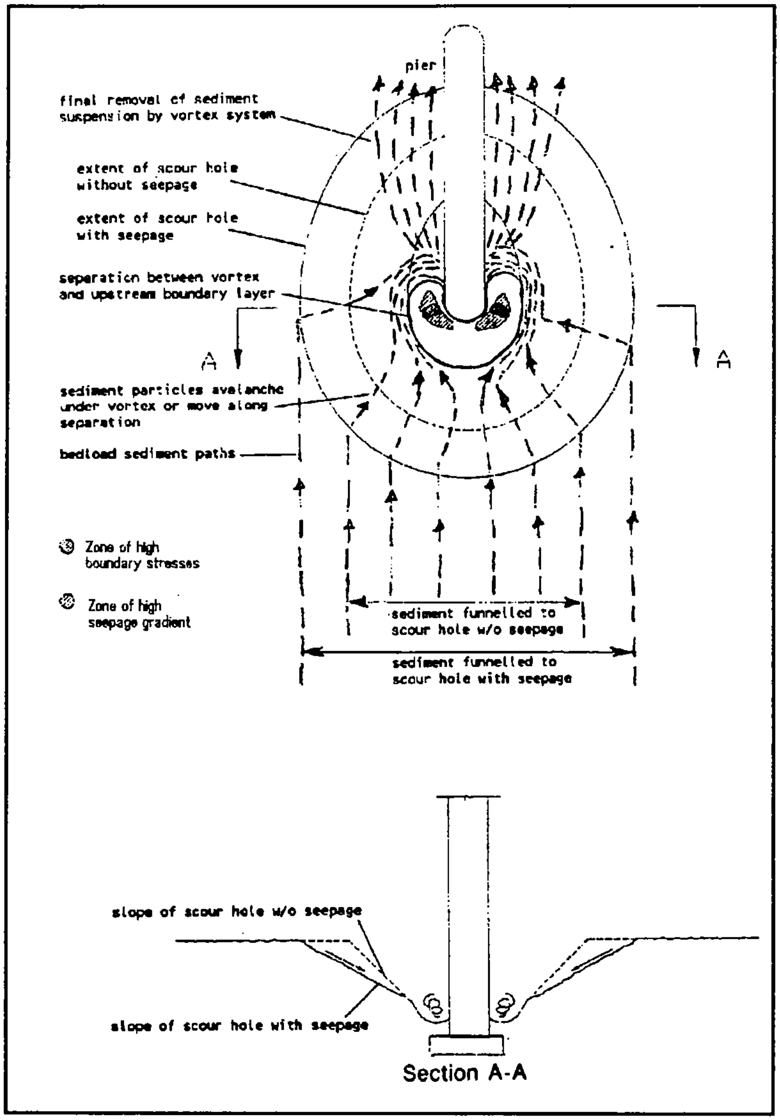

Figure 8: Increase in sediment transport to scour hole due to inflow seepage.

layered sediments is probably negligible.

The effect of seepage caused by backwater effects on constriction scour may be appreciable, especially when pressure flow exists (upstream water surface elevation exceeds superstructure elevation), as shown in Figure 2b. Upstream of the bridge, seepage is influent (into the bed and banks), while underneath and downstream of the bridge, seepage is effluent (out of the bed and banks). The influent seepage increases fluid drag on particles and causes the channel boundary layer to be reduced. Both effects increase the boundary stress just upstream of the bridge superstructure. Underneath the superstructure, the effluent seepage is likely to increase the thickness of the channel boundary layer and cause a reduction in boundary stress. The effluent seepage in this region would tend to increase the channel boundary layer and, as a result, decrease the potential for erosion; however, this decrease in potential does not mean that deposition will occur. The channel velocities continue to increase through the constriction in the downstream direction. The decrease in erosion potential due to effluent seepage may not be significant compared to the increase in erosion potential caused by increased velocity coupled with the destabilizing uplift seepage forces acting on particles.

Effects of Seepage on Local Scour

Local scour is influenced by sediment transport characteristics of the flow and the velocity of the approaching flow. General inflow seepage affects bed forms, bed roughness, average velocity, and the stable slope angle. Changes in bed roughness and average velocity influence the velocity profile. Changes in velocity profiles affect the strength of local vortex systems. Richardson et al. (1985) showed that seepage into the stream caused increases in average flow velocity. Velocity increases would result in increases in erosion potential of local vortex systems; however, Abt et al. (1988) found that increases in inflow seepage cause a reduction in scour hole depth of up to 60%. In addition, the researchers found that scour hole lateral extent was increased by over 200%. The reduction in scour depth and the increase in scour hole extent with inflow seepage can be explained by considering the stable slope angle of the scour hole and the change in sediment transport to the scour hole. Without seepage, the scour hole increases in depth until the amount of sediment captured by the scour hole is equal to the erosion capacity of the local vortex systems. The amount of sediment captured by the scour hole is a direct function of its lateral extent. The lateral extent is largely dependent on the depth of scour and the stable angle of the scour hole material. Inflow seepage causes the slope angle of the scour hole to decrease, resulting in an increase in the extent of the scour hole and a significant increase in sediment captured by the scour hole from the upstream flow (Figure 8). Apparently, the increase in average velocity due to seepage into the stream is small compared to the increase in sediment transport to the scour hole caused by the reduction in stable slope angle of the scour hole.

Inflow seepage caused by backwater effects may affect scour hole development in varying ways, depending on the geometry of the pier with respect to the superstructure and embankments. Changes in slope angle of the scour hole will result; however, the exact changes are difficult to predict. In the upstream portions of the scour hole, seepage into the bed is likely to occur and result in the steepening of the scour hole, while in the downstream portion of the scour hole, seepage into the stream is likely to result in scour hole widening.

Inflow and outflow seepage caused by local flow accelerations are an integral part of the local scour process as described previously. The seepage gradients in close proximity to piers and abutments are likely to be larger than at any other locations in the streambed (Hjorth 1975). These large hydraulic gradients are likely to influence erosion in most streambed materials.

Seepage Transience and Bridge Scour

Although Ettema (1980) illustrated the time-variant nature of scour around a pier and Hjorth (1975) showed the pulsating character of vortices and pressure differences around a pier, neither these investigators nor any of their colleagues have conducted experiments that truly simulate the seepage-scour interplay around piers in natural streams. As stage increases during a flood event, the pressure in the stream exceeds porewater pressure and water begins to seep into the bed. Scour initiated by tractive force mechanisms and by fluctuating vortices and pressure differences occurs in a setting of general seepage into the stream bed. Such flow into the bed would have an overall stabilizing influence on the side slopes of the scour hole. As pressure in the bed porewater became equal to stream pressure, this stabilizing seepage would wane and scour hole slopes would gradually fail; the observable effect would be a widening of the scour hole. This process would occur even in the absence of continued sediment transport. When the stream stage began to decrease, the pressure in the porewater would become higher than the stream water pressure, and general area seepage outflow would occur from the stream bed. This general outflow may be much less intense than the pulsating outflow that plucks particles out of the bed as the scour hole forms, but it occurs over a much larger area. The net effect of this outflow is a destabilizing seepage stress in the side slopes of the scour hole. Figure 9 shows a hypothetical sequence of changes at a pier, caused by combined scour and seepage effects. When the outflow seepage occurs in uniform bed materials, the scour hole may fill by slumping of the side walls, as do some stream banks (Thorne et al. 1988), or by the complex zoned process described by Howard and McLane (1988). The net result is a "scour" feature much shallower and wider than the initial scour hole formed during high early discharge. It is important to note that a progressive failure by piping or tunnel scour into the side walls of the scour hole may occur long after the flood has receded; bed soil hydraulic conductivity greatly influences the rate of progressive failure. The widening of the scour feature may initiate instability in the adjacent streambank with consequent detrimental effects on supported abutments, perhaps long after the flood event. The final slope of the scour hole side walls is a function of the relative density of the soils in the bed if the bed consists of cohesionless materials, as well as of the hydraulic conductivity of the bed. The rate of progressive failure depends on the seepage flow rate; the failure process is a grain-by-grain removal. The intensity of the removal is governed by the hydraulic gradient driving the outflow seepage; in very pervious beds, outflow gradients may not be sufficient to cause this type of failure. Consideration of this mechanism may explain Neill's (1976) observation of unanticipated depth of scour holes in sand-bed streams and failure of holes to fill after flood events on gravel-bed rivers. As Neill concluded, "In the case of sand-bed rivers, little reliance can be placed on low-water observations of maximum scoured depths...," and in "...many coarse gravel-bed rivers, the scour holes are relatively stable and do not fill in at low stages." Outflow seepage in sands would cause scour hole collapse and in-filling, whereas outflow gradients in gravel would be low, the gravel would have high shear resistance, and the seepage pressures would equilibrate quickly, so no delayed failures would occur. The change in the side wall slope of a scour hole can be very significant; the slope angle may be halved. In Harrison's tests (1970), the slope angle of the downstream limb of bed dunes changed by 19 degrees when bed seepage was changed from inflow to the bed to outflow into the stream. Taylor (1948) showed how the slope angle in cohesionless soils would be reduced by seepage effects, as mentioned previously.

Possible interactions between scour hole widening and bank erosion and abutment collapse are described in the following section on seepage influences on bank and

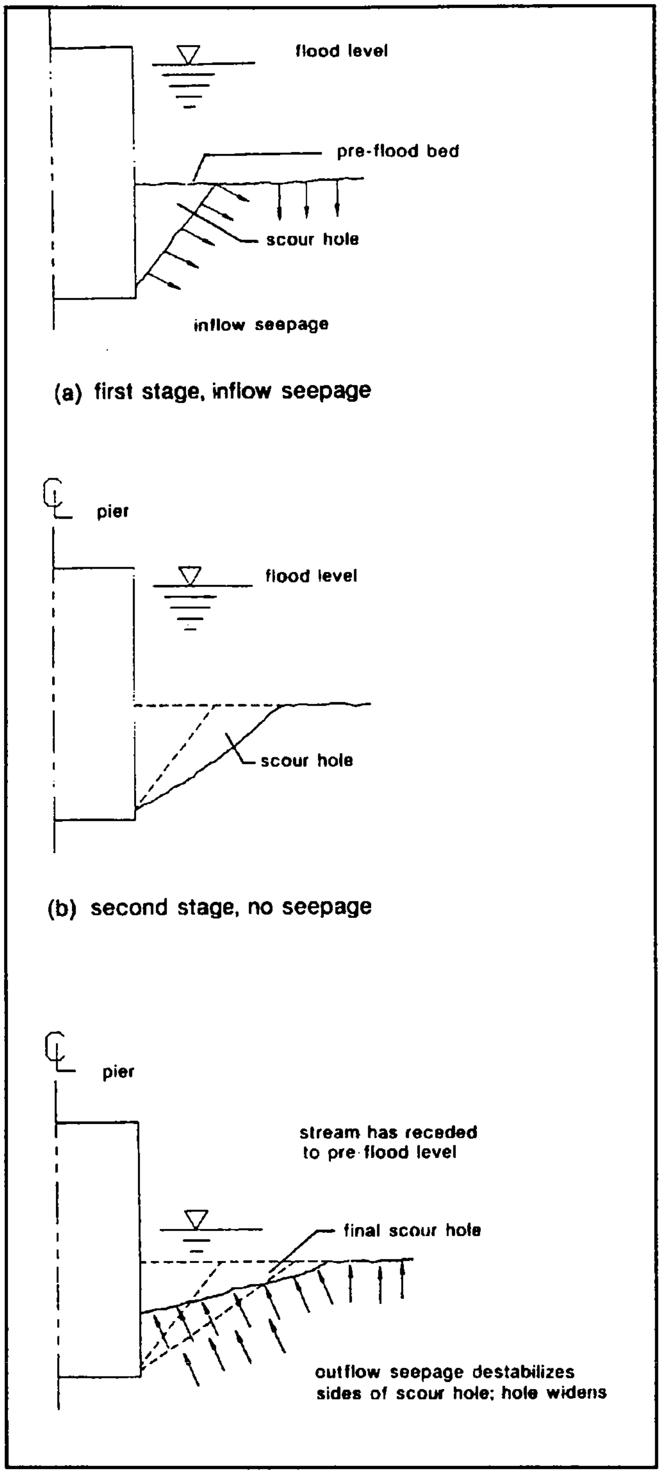

Figure 9: Sequence of scour hole formation.

abutment stability. Before that treatment is presented, however, it is necessary to note another way in which most experimental studies have not simulated actual conditions in alluvial streams. Most scour experiments have been flume studies on behavior of homogeneous bed materials; most alluvial stream beds consist of heterogeneous layers and lenses. Not only do these layers possess greatly differing resistances to tractive scour (sand versus silt versus clay, as illustrated in the common Shields diagram), but also seepage effects differ in these materials because of differences in shear strength and hydraulic conductivity among them. The significance of such differences can be shown perhaps most readily in the description of seepage effects on layered banks that follows.

SEEPAGE INFLUENCE ON ABUTMENT SCOUR/BANK EROSION

If an abutment significantly constricts flow in a stream, scour may occur at the abutment by much the same mechanisms that cause scour around piers. Tey (1984) showed how a vortex would form at an abutment at the junction of the upstream wing wall and the main abutment wall and how a scour hole would be initiated in the bed at that point. However, he did not postulate any significant ejection of bed grains in the wake zone or at the junction of the main abutment wall and the downstream wing wall. Taylor (1978) has shown how erosion just downstream from pier-abutment alignments can be very significant. Tey's interpretation is at variance also with the results of Hjorth and Ettema. Undoubtedly, pressure differences would be created adjacent to the abutment as they would near a pier; an abutment with wingwalls can be considered analogous to half of a long, sharp-nosed pier, with flow divided cleanly at the nose. In the case of an abutment, the overall stability is influenced by stability considerations of the bank on and in which the abutment is constructed. Constriction of the stream at a bridge undoubtedly can lead to increased recharge of water into the banks at the bridge, compared to the site without the bridge structure. The interaction of seepage outflow and tractive force scour obviously is complex, as shown by the preceding description. The seepage outflow may be concentrated by variations in hydraulic conductivity in bank materials (Okagbue and Abam, 1986). Seepage outflow into scour holes may widen the zone of instability so that the toe of the bank or abutment slope is undermined, but the undermining may occur long after the crest of the flood is reached. Seepage outflow effects on bed scour around piers may not be appreciated because the bed particle ejection and entrainment may occur in turbid water during floods and the resultant scour holes may collapse and partially fill as a result of general seepage outflow before the water in the stream slows and clears. In exposed stream banks, failures may occur long after flood events because of the slow rate of seepage outflow (Ullrich et al. 1986) and may not be obvious because of low flow rates out of silty soils; nevertheless, piping/sapping has been observed in silty layers between clay strata (Hagerty and Spoor 1989). Also, when a stream rises against its banks and high velocities furnish ample energy for tractive scour, bank erosion may be severe because the bank is covered with an irregular array of broken and fallen slabs and blocks as well as loose accumulations of sandy particles sapped from the bank. Local turbulence among these slabs and blocks of disturbed soil and loose piles of sand will be much more effective in removing bank materials than would the currents created by the same discharge between smooth banks of undisturbed soil. Outflow seepage also can cause failure in banks over which an effort has been made to provide protection against tractive scour forces. More than 80 local protection projects along the Ohio River were inspected in 1988; the riprap system had failed in almost all cases where riprap had been used over an eroding bank without an intervening transition layer. As part of that same inspection, numerous launch ramps were evaluated; many of the concrete ramp slabs were found undermined by seepage outflow because no transition zone had been placed over the graded stream bank before ramp construction was begun. The same processes that caused failure in the inspected riprap layers and ramps could cause failure in bridge abutments on alluvial soils. It should not be inferred, however, that protection cannot be provided against scour and seepage outflow effects.

PREVENTION OF FOUNDATION SCOUR AT PIERS/ABUTMENTS

Because of the locally high boundary stress produced by the vortex systems at piers and abutments, rock sizes required to protect the streambed surrounding the bridge piers and abutments are several times larger than the rock sizes required to protect unobstructed streambeds. On the basis of small-scale model studies, Parola and Jones (1991) have developed a relation for determining the size of riprap sufficient to protect streambeds surrounding bridge piers. Similar experiments were conducted by Pegan (1991) for riprap protection of square abutment walls and spill-through abutment embankments. However, provisions must be made to prevent "leaching" of fine bed material through the riprap protection. Posey (1974) conducted small-scale model experiments on riprap protection and found that riprap placed on a sand bed would sink to a level beneath the streambed approximately equal to the scour depth that was observed for the same flow conditions without riprap protection. Posey (1974) advocated the use of an inverted filter beneath the riprap to prevent "leaching" of the fines around riprap, allowing it to sink.

To accommodate the effects of seepage outflow on bed and bank erosion, it is necessary to understand that seepage effects include 1) particle ejection and entrainment caused by pressure fluctuations around piers and abutments when velocities are significant, and 2) general instability associated with seepage outflow from bed and banks when stream stage drops, as a result of slumping and/or piping/sapping mechanisms. Construction of a weighted graded filter or use of a filter fabric anchored with an overlying coarse protection layer can provide protection for piers (Posey 1974) and stream banks (Heerten and Wittmann 1985). Use of filter zones below large-stone protection layers has been shown to be effective. In many installations, very expensive large riprap has been used in misguided efforts to counteract "severe scour" problems, when much smaller stone used with an appropriate filter could have provided much more adequate protection at much lower costs. (Cutter and Waterman 1984).

CONCLUSIONS

Scour at bridge piers and abutments is caused by a combination of tractive drag forces and seepage outflow effects. The vortices around piers and the pressure differences that cause outflow seepage during floods fluctuate and pulse. Similar vortices and seepage effects can be expected around abutments. Porewater pressures in the bed and bank also can cause general area outflow, which can collapse and fill scour holes as well as undermine banks and abutments. These mechanisms are progressive; the time-variant and progressive nature of seepage outflow effects complicate the process by which failure occurs. Delayed failures, or failure only after several repetitions of given discharges, may be explained by consideration of the time-dependent character of seepage outflow effects. Further complexity is introduced into the failure process by the layered nature of alluvial soil deposits. Protection against these mechanisms can be achieved; however, design of effective but efficient protection systems requires understanding of the complexities of the failure process. Much more research is needed, including experiments in which seepage occurs in a time-variable pattern like seepage in real streams, and much more field and laboratory effort must be devoted to observing the response of layered soils to combined scour-seepage forces.

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MEASUREMENT OF SCOUR AT SELECTED BRIDGES IN NEW YORK

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ABSTRACT

The U.S. Geological Survey (USGS), in cooperation with the New York State Department of Transportation (NYSDOT), collected bridge-scour data at 77 sites throughout New York, excluding Long Island. This report is part of a six-year study to evaluate data-collection methods and predictive equations for local scour at bridges. Methods are similar to the "limited approach" developed by the USGS in cooperation with the Federal Highway Administration.

One to two feet of local scour was found at many sites at the start of the study. At a few sites, the scour has exposed spread footings that were buried during construction. Fifteen high-flow measurements, including two flows with a recurrence interval exceeding five years, did not indicate any new scour beyond the existing holes. mows with recurrence intervals greater than five years may be necessary to trigger scour in streams with coarse bed material.

Sonar and geophysical techniques were evaluated for their effectiveness in bridge-scour investigations. A transducer inside a 100-pound sounding weight provided an alternative method of measuring water depth, although moving the unit across the bridge was cumbersome. Another method used a transducer, installed on a bridge pier, and a data logger that recorded changes in streambed elevation automatically at selected time intervals. Geophysical techniques applied to gravel and cobble streambeds did not detect any backfilled scour holes, possibly because (1) holes did not exist, (2) resolution of the equipment (1 to 2 feet) could not detect a shallow infilled layer, or (3) the infilled material was the same as the streambed.

INTRODUCTION

Approximately 500,000 bridges in the United States are built over water and are subject to scour, the most common cause of bridge failure. Accurate estimates of potential scour are needed to design, construct, and maintain bridges. The added cost of making a bridge resistant to scour is usually small compared to the cost of bridge failure (Federal Highway Administration, 1988).

Floods in June 1972 damaged 182 bridges along New York State roads and many bridges on county roads. Scour and debris were the primary causes of damage (Highway Research Record, 1973). Damages from floods in April 1987 ranged from abutment washouts of short, single-span bridges over small streams to the catastrophic collapse of the five-span, multilane New York State Thruway bridge over Schoharie Creek that claimed 10 lives (Zembrzuski and Evans, 1989). Floods in June 1989, in western New York, damaged several bridges.

In 1988, the USGS, in cooperation with the NYSDOT, began a six-year study of bridge scour in New York through methods similar to those used in its national bridge-scour program in other States. The objectives were to (1) compile a statewide data base, (2) evaluate data-collection methods and predictive equations for local scour, and (3) identify the types of channels and bridges that are vulnerable to scour. This report describes the techniques used to collect bridge-scour data at 77 sites and presents the criteria for site selection, methods of data collection, and types of equipment used. It describes, in general terms, the extent of scour and discusses the limitations of certain procedures and equipment. It also presents a comparison of conventional methods of data collection with sonar and geophysical techniques.

TYPES OF BRIDGE SCOUR

Scour is the erosive action of flowing water that removes material from the streambed (Federal Highway Administration, 1988), and *scour depth* is the depth to which material is removed below a stated datum. Scour is a natural phenomenon that occurs in alluvial streams; it also can occur in any stream that contains erodible bed material.

Three types of scour can occur at a bridge: general scour, constriction or contraction scour, and local scour. *General scour* is the progressive degradation of the streambed from natural or artificial processes in a channel over many years. *Constriction scour* is streambed erosion caused by increased flow velocities near a bridge that results from the decreased flow area formed by the bridge, approach embankments, or piers. *local scour* is streambed erosion caused by local disturbances in the flow, such as vortices and eddies in the vicinity of piers. A general practice in bridge design is to estimate the depth of each type of scour separately, then sum the estimated depths to obtain the total scour depth. Local scour may produce greater scour depths than the other types of scour and is the primary focus of this study (Richardson and others, 1988).

The depth and extent of scour is determined by the following factors, described by Richardson and others (1988), Raudkivi and Ettema (1983 and 1985), Kingeman (1973), and Blodgett (1984):

- Velocity, depth, and angle of approach flow
- Size and gradation of bed material
- Bridge geometry
- Presence of debris or ice
- Duration of high flow
- Channel geometry
- Total number of high flows
- Channel morphology

The flow pattern and vortex system induced by a pier are shown in figure 1. These vortices result from the pileup of water at the upstream face and subsequent acceleration of the flow around the nose of the pier. Two types of local scour are "clear-water" scour and "live-bed" scour. Clear-water scour occurs when bed material upstream of the scour hole is motionless and cannot replace material removed by scour, and scour depth increases if the scour mechanism is able to remove material from the hole. Live-bed scour occurs when bed material upstream of the scour hole is moving, and scour depth increases only if the removal rate of material from the hole exceeds the transport rate of material into the hole.

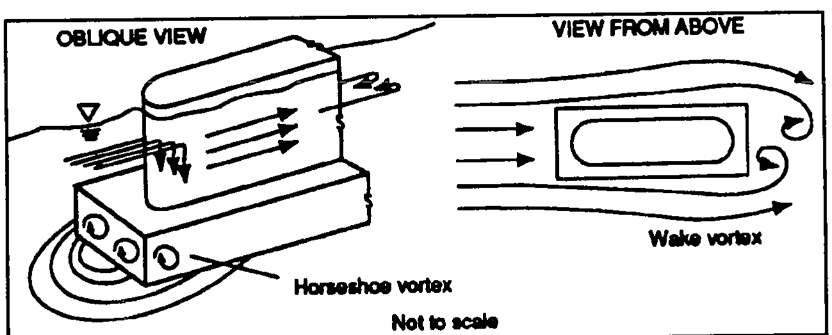


Figure 1: Flow pattern and vortex system induced by a pier. (Modified from Richardson et al, 1988)

MEASUREMENT OF SCOUR

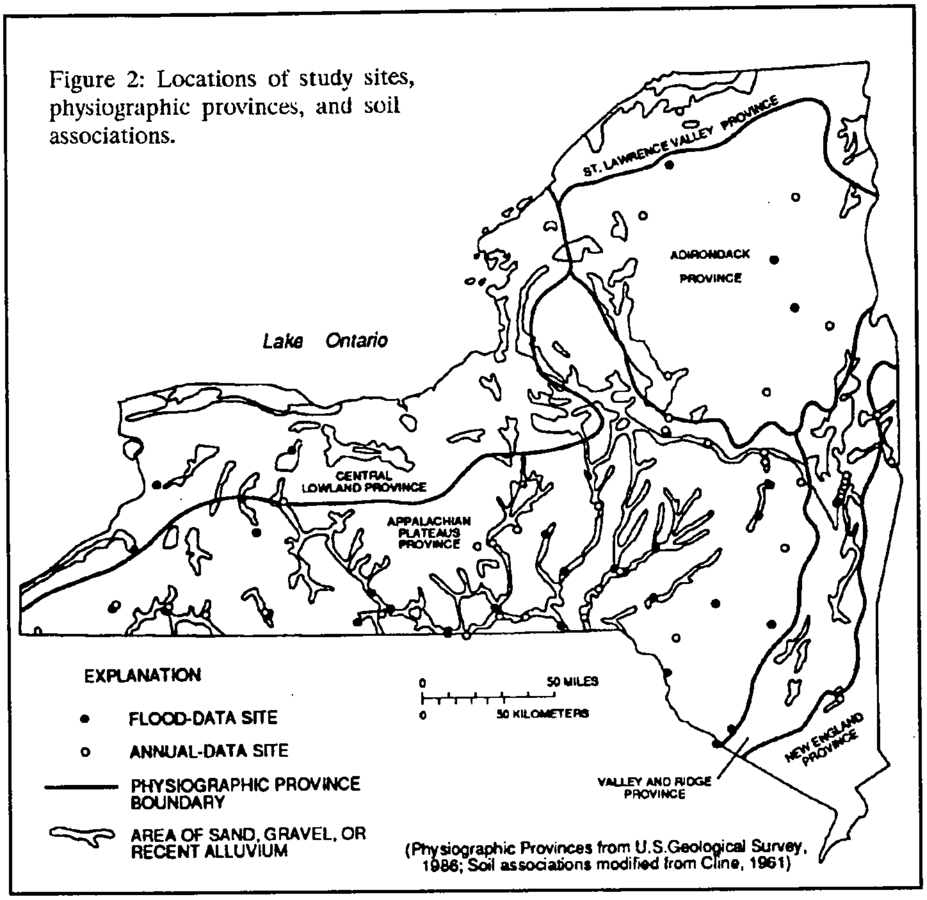
The dynamic processes of a stream can cause the streambed to degrade and then aggrade during the course of a flood. Scour holes may develop and fill before the stream returns to normal levels. The interface between the backfilled material and the scour hole can be measured by geophysical techniques if the two layers have differing electrical or seismic-reflection properties (Gorin and Haeni, 1989).

Turbulence during floods in New York has caused 150-lb weights to skip along the water surface; this downstream movement caused errors in the depth measurement. Although corrections can be applied to compensate for most of this type of error, the exact location of the weight is always uncertain (Rantz and others, 1982; Coon and Futrell, 1986; Beverage, 1987). The use of mobile and fixed sonar instruments to measure scour depth is being studied. The mobile technique is similar to the method used by the USGS in Arkansas (Southard, 1989), where a graphic recorder plots a cross section of the streambed while a transducer, submerged 1 to 3 ft, is moved across the stream. A fixed sonar installation automatically records streambed elevation at the base of a bridge pier.

Reconnaissance

An extensive literature search provided many articles on scour. Field data were sparse, but USGS and NYSDOT files provided some information related to scour. Stage-discharge ratings for USGS gauging stations were analyzed to determine channel stability near the bridge. Many rating curves at a station may indicate bed-material movement. Data from crest-stage gauges and partial-record sites were also reviewed, and stations in areas of erodible bed material (sand, gravel, or recent alluvium) were identified (fig. 2). Stations on streams

Figure 2: Locations of study sites, physiographic provinces, and soil associations.



with drainage areas greater than 100 mi^2 and a potential for scour also were identified. Factors that affect scour potential include erodible bed material, high stream velocity, and any documented scour nearby. Bridges with a medium or high scour-susceptibility rating in NYSDOT bridge-inventory file were reviewed, and bridges scheduled for immediate scour countermeasures (riprap, concrete-filled bags, etc.) were excluded.

Site selection

The locations of bridge sites studied are shown in figure 2. A total of 77 bridge sites were selected--31 for flood-data collection and 46 for annual-data collection. The network represents six physiographic provinces in upstate New York and includes a wide range of basin characteristics and bridge designs. Drainage areas range from 30 mi^2 to more than 8,000 mi^2. Few gauging stations met the selection criteria in the Central Lowland, St. Lawrence Valley, and New England physiographic provinces. All bridges were constructed between 1902 and 1989.

Sites identified from reconnaissance were visited for evidence of scour. A checklist developed by the USGS to standardize the selection process is depicted in figure 3. If a bridge did not meet the selection criteria, the next two bridges upstream and downstream of the site were visited.

Study sites were divided into two categories: flood-data sites and annual-data sites. Flood-data sites represent locations where data are to be collected during high flows; data from these sites can be used later to determine what types of channels and bridges are vulnerable to scour and to evaluate local scour equations. Annual-data sites provide an inexpensive method of expanding the data base; at these sites the streambed elevation along the upstream side of the bridge is measured annually. Priority was given to sites near USGS gauging stations along streams that contain erodible bed material or appeared, from review of USGS rating curves and NYSDOT files to be unstable.

The site-selection criteria were as follows:

- Site is at or near a USGS stream-gauging station to facilitate data collection.
- Drainage basin should exceed 100 mi^2. Smaller basins generally contain single-span bridges (no piers), and the short duration of high flows limits the scour mechanism and the ability to collect flood data.
- Streambed contains an ample supply of bed material prone to scour. Piers on bedrock or protected by riprap are excluded.
- Pier nose is square, round, or sharp.
- Network represents a wide range of basin characteristics.
- Pier is in the main channel.
- Channel is uniform upstream and downstream from bridge.
- mow-angle approaching pier is 10 degrees or less.
- Scour is evident (although having a few sites with no

Rating Item	+	0	-
Is bridge accessible at high flow? Yes (+); No (-)			
Is streambed composed of bedrock or clay? No (+); Yes (-)			
Distance from bridge deck to streambed (in feet)? Less than 40(+); 40 to 80(0); more than 80(-)			
Is sustained high flow likely during a flood? Yes (+); No (-)			
Can scour be measured safely at this bridge? Yes (+); No (-)			
Are there any other factors that would prevent scour from being measured at this site? No(+); Yes(-)			
Is scour likely to occur at one or more piers? Yes (+); No (0)			//////
Is scour likely to occur at more than one pier? Yes (+); No (0)			//////
Is scour likely to occur at one or more bridge abutments? Yes (+); No (0)			//////
Can pier be reached by a sounding weight lowered from the bridge? Yes (+); No (0)			//////
Does the bridge constrict high flows significantly? Yes (+); No (0)			//////
Shape of pier nose: square or round (+); sharp (0)			//////
Angle at which flow approaches piers (in degrees): 0 to 5 (+); more than 5 (0)			//////
Are pier footings exposed? No (+); Yes or don't know (0)			//////
Has riprap been placed around one or more piers? No (+); Yes or don't know (0)			//////
Is debris lodged on one or more piers? No (+); Yes (0)			//////
Is a gaging station located nearby (within view of the bridge)? Yes (+); No (0)			//////
Is boat access available nearby? Yes (+); No (0)			//////
Does the bridge have trusses? Yes (+); No (0)			//////
Will a traffic lane need to be closed to make measurements? No (+); Yes (0)			//////
Totals (+, 0, and -)			

Figure 3.--Sample checklist for bridge-site selection.

scour is acceptable).

- Bridge does not encroach upon main channel.
- Pier does not constrict cross-sectional flow area by more than 10 percent.
- Nearest reservoir is at least 10 mi upstream from the site.
- Quantity of debris or ice is minimal.
- Water depth at a few piers always exceeds 5 ft.
- Boat access is available on large streams (to facilitate data collection).
- Information on site construction, inspection, and maintenance is available.

Additional criteria for flood-data sites are:

- Scour hole is accessible from upstream side of bridge.
- Distance from bridge deck to streambed is less than 80 ft, preferably less than 40 ft.
- Bridge is wide enough to provide safe working space for a two-person crew and measuring equipment, and does not interfere with operation of equipment.
- Telemetry is available or observer is nearby to provide flood-alert information.

Data Collection

Data collection began in May 1989 at 77 bridges in New York (fig. 2); high-flow data are being collected at 31 bridges, and annual cross-section data at the remaining 46. Methods are similar to the "limited approach" developed by the USGS in cooperation with the Federal Highway Administration, whereby discharge, velocity, streambed elevation, and bed-material data are collected through equipment and procedures compatible with the survey's stream-gauging program (Jarrett and Boyle, 1986). This approach is being used in the USGS's national bridge-scour program and in similar studies in other States; scour data collected in these studies may supplement data collected in New York. In addition, geophysical and sonar techniques are being used at a few sites to evaluate their effectiveness.

Flood Data.--This information is used to identify changes in streambed elevation, velocity distribution around piers, and bed-material characteristics. Data are used to determine the types of channels and bridges vulnerable to scour and to evaluate local scour equations.

Reference points were established at four cross sections per site--the upstream and downstream bridge railings, the approach section (one bridge-width upstream), and the exit section (one bridge-width downstream). The water-surface and streambed elevations at each cross section are calculated from the reference-point elevation.

Copies of bridge plans, drill logs, maintenance and inspection sheets, and fathometer surveys were obtained from NYSDOT. Dimensions of the piers and footings were recorded from bridge plans or site inspections. Channel-roughness coefficients were estimated at each cross section.

Recurrence intervals of the two-year (mean annual) and five-year floods were estimated from guidelines outlined by U.S. Water Resources Council (1981) or multiple regression analyses (Zembrzuski and Dunn, 1979). Flood data are to be collected whenever streamflow exceeds the mean-annual recurrence interval. Post-flood data are to be collected if the flow exceeds a five-year recurrence interval. Intervals were based on studies of sand, gravel, and cobble streambeds in which thresholds for particle motion were exceeded during flows of these magnitudes (Culbertson and others, 1967; Norman, 1975; Andrews, 1979; Andrew, 1979; and Sidle, 1988).

Bed-material samples were collected at the water's edge near the bridge. A variation of the grid sampling technique (International Organization for Standardization, 1989) is used because the streambeds are armored. The intermediate axis of each stone is measured every 0.5 feet along a 50 foot tape. The frequency of each size interval is the percentage, by number, of the 100 stones in the original sample that fall in the interval. A USGS bedload sampler is to be used at streams that could experience live-bed scour at high flow to determine the size of the bed material in motion (Helley and Smith, 1971).

The size distribution of the subsurface material is estimated from a 5- to 10-lb bulk sample collected after removal of the armor layer. The frequency of each size interval is expressed as the percentage, by mass, of the original sample that falls within the interval. The relations among differing methods of sampling that have been established for densely packed cubes in random arrangement indicate that the grid sample (by number) frequency is equivalent to the bulk sample (by mass) frequency (International Organization for Standardization, 1989). Core borings are to be taken at selected sites for comparison with data collected by grid and bulk sampling methods.

Baseline cross sections were measured at each reference point at the beginning of the study to determine the extent of scour. Streambed elevations are to be compared with (1) those shown on bridge plans, (2) previous measurements at the site, and (3) data collected during the study. About 20 soundings were used to define each cross section, and additional soundings within one "pier width" of each pier were used to define the streambed at the upstream and down-stream sides of the pier. The cross sections at the approach- and exit sections are used to measure general scour; those at the bridge are used to measure constriction scour; and soundings near the pier are used to determine local scour.

Whenever discharge at a site exceeds the mean-annual flood, the following procedures are to be used:

1. Make a standard discharge measurement at the upstream side of the bridge.

2. Measure gauge heights at the approach section, upstream and downstream sides of the bridge, and exit section before and after the discharge measurement.

3. Make depth soundings at the upstream and downstream sides of the bridge by the standard depth measurement with a sounding weight or the mobile-sonar technique. If the standard method is used, make the soundings about 1 foot apart within one "pier width" of the pier, and remove the velocity meter to reduce drag.

4. Photograph the stream to document the hydraulic conditions, particularly the state of flow, direction of flow approaching bridge, presence of debris, eddies, water-surface pileup, and drawdown.

5. Evaluate bedload-transport conditions with a bedload sampler or by listening for the sound of rocks striking the bridge or other rocks.

6. Measure water temperature.

7. Make depth soundings and measure gauge height at both sides of the bridge after the flow recedes if the recurrence interval of the flood exceeds five years to determine whether changes in streambed elevation have occurred.

Annual Data.--This information is used to expand the

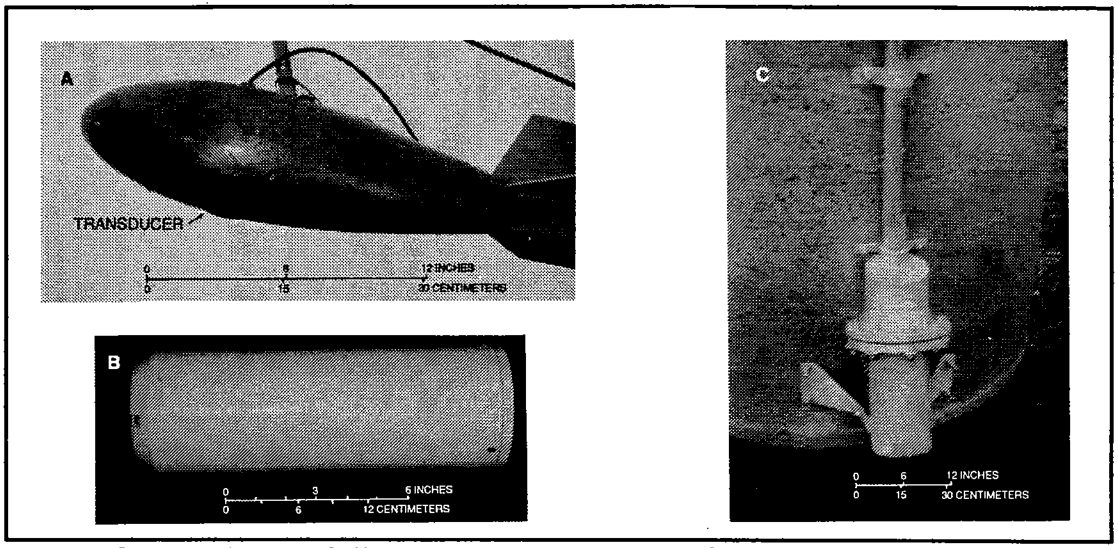


Figure 4: Sonar equipment. **A.** Transducer in 100-lb weight. **B.** Sonar unit at fixed installation. **C.** Protective shield for unit, mounted onto bridge pier.

data base along the upstream side of a bridge. The streambed elevation is measured annually in relation to a reference point established on the upstream side of each bridge. Dimensions of the piers and footings are determined from bridge plans or site inspections.

Supplemental Data.--Since 1984, NYSDOT bridge inspection procedures require recording of scour depth every 2 years, and diving inspections at bridges in deep water every 5 years. This information, along with data from bridge plans and USGS measurements, are to be analyzed to determine long-term changes in streambed elevation.

Equipment.--Standard USGS streamflow-measuring equipment is used to collect most of the scour data. This equipment includes a bridge crane, velocity meter, and sounding weight (50 to 100 lb); descriptions are given in Rantz (1982). Fathometers mounted on boats, floats, and piers have been used to measure scour (Norman, 1975; Hopkins and others, 1980; and Skinner, 1986). In this study, sonar and geophysical equipment is being tested for ease of operation, safety, and accuracy. The equipment must be reliable, simple, and practical.

Four types of geophysical equipment are being used to analyze scour: ground-penetrating radar, tuned transducer, color fathometer, and black-and-white fathometer; descriptions are given in Gorin and Haeni (1989). The radar system, with dual 80 Mhz (megahertz) antennae, is floated in water 5 to 20 feet deep. The output is recorded on magnetic tape and a graphic recorder. The tuned transducer operates from a boat at a frequency of 3 to 14 Khz. The output is recorded with equipment similar to the radar system. The color fathometer, operating at 20 to 100 Khz, digitizes reflected seismic signals and assigns a color for every 6-decibel change in the acoustic impedance of the reflected signals. The output is recorded on cassette tape and displayed on a color monitor. A black-and-white fathometer, operating at a frequency of 200 Khz, can distinguish only between a hard and soft streambed. This system provides a rapid and accurate depth measurement and is used with the other geophysical equipment to verify the water depth. Output from the fathometer is plotted on a graphic recorder.

The mobile sonar system is identical to the black-and-white fathometer except that the transducer is mounted in a 100-lb sounding weight instead of in a boat (fig. 4A). The design of the sounding weight enables the transducer to remain horizontal in the water.

A sonar unit (fig. 4B) attached to a bridge pier is being evaluated. This system was chosen because the analog output can be transmitted to a data logger as far as 300 feet away; other systems with digital or graphic output were not compatible with a data logger and must be within 100 feet of the transducer. The force of waterborne ice and debris can severely damage or destroy this equipment; therefore a shield was designed to protect the transducer and microprocessor (fig. 4C).

Newer models allow the microprocessor to be placed out of water. The data logger activates the unit at preselected time intervals and transmits the data by satellite telemetry.

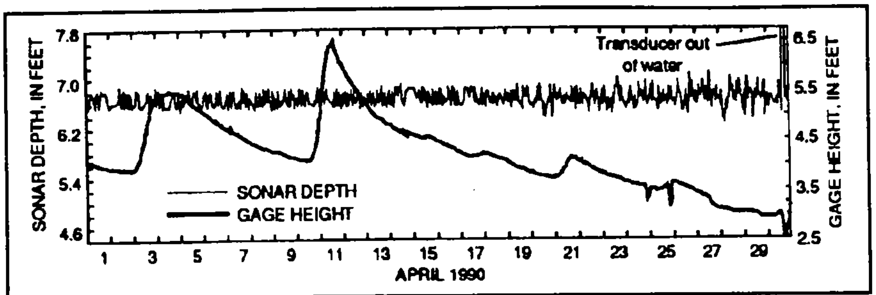


Figure 5: Sonar and gage-height output from data logger.

Observed Scour at Selected Sites.--One to two feet of local scour was found at many sites at the start of the study. At a few sites, the scour has exposed spread footings that bridge plans show to have been buried during construction. Many of these holes may have been caused by clear-water scour during a previous flood or floods. Fifteen high-flow measurements, including two flows with a recurrence interval exceeding five years, show no additional scour since the initial observation. These results agree with Sidle (1988) that scouring of coarse material is triggered by flows with recurrence intervals greater than five years.

LIMITATIONS OF PROCEDURES AND EQUIPMENT

One objective of the study is to evaluate the accuracy, safety, and ease of operation of the procedures and equipment. Stream velocity and depth are difficult to measure near piers in deep, swift streams, especially when debris is present, and heavy weights (100 to 150 lb) are not always adequate to stabilize the equipment. When mobile- and fixed-sonar installations are used to measure water depth, air or sediment entrained in the flow may interfere with the signal. Also, even though the mobile equipment can be brought to a site rather than installed permanently, moving it across the bridge and recording data have been found cumbersome. Fixed installations, by contrast, can automatically record streambed elevation at selected time intervals but must be extremely rugged. A plot of the output from the data logger is shown in figure 5. Signal scatter, due to wide reflections from cobbles, increases as the signal ground loses contact with water (gauge height 4.0 ft), and spikes or "lost signals" occur when the transducer is exposed to air (gauge height 3.0 ft). This equipment has been tested for one year, in which the peak flow had a recurrence interval less than the mean-annual flood, and no scour was observed.

Geophysical techniques were applied to gravel and cobble streambeds but did not reveal any backfilled scour holes. Probable reasons are that (1) holes did not exist, (2) resolution of equipment (1 to 2 ft) did not permit detection of a shallow infilled layer, or (3) the infilled material was the same as the streambed. The usefulness of geophysical techniques depends on the characteristics of the site. The equipment is sophisticated and requires a high degree of skill for effective operation and interpretation. Many objects can interfere with the signal; for example, buried pipes, rocks, backfill from construction, and side echoes. The most useful results are likely to be from streams that undergo live-bed scour.

The number of years of available record and hydrologic conditions during the sampling period may limit the amount of data collected in some basins. If scour countermeasures are installed at some sites by NYSDOT before the project is completed, results will be affected.

SUMMARY

Scour data are being collected at 77 bridges in New York, excluding Long Island. Bridges near USGS gauging stations on streams with erodible bed material were selected in six physiographic provinces. High-flow data are being collected at 31 bridges, and annual data at the remaining 46 bridges. The conventional method of data collection by a sounding weight is being compared with sonar and geophysical techniques for ease of operation, safety, and accuracy. Cross sections measured at the beginning of the study are to be compared with bridge plans, previous measurements, and data collected during the remaining years of the project to determine the extent of scour in New York.

One to two feet of local scour was found at many sites at the start of the study. At a few sites, the scour has exposed spread footings that bridge plans show to have been buried during construction. Fifteen high-flow measurements, including two flows with a recurrence

interval exceeding five years, did not show any new scour. Present scour holes and the coarse bed material may indicate clear-water scour is more common than live-bed scour.

Geophysical techniques were applied to gravel and cobble streambeds but did not reveal any backfilled scour holes. The effectiveness of these techniques depends on local conditions, and the methods and equipment require a high degree of skill for effective operation and interpretation. Streams with fine bed material probably provide more useful results than those with coarse material.

A fathometer provided quick and accurate depth measurements. The mobile method was cumbersome, and the fixed installation required extensive protection. A fixed installation designed to record streambed elevation automatically at selected time intervals is expected to provide useful information during floods. Further study is needed to determine how well these units operate amid flood turbulence, debris, sediment, and ice.

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GROUND PENETRATING RADAR STUDY OF "BRIDGE SCOUR" IN NEW YORK STATE

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ABSTRACT

Results of the Ground Penetrating Radar Inspection Program at Clarkson University are presented and discussed with respect to the causes, extent, and results of riverbed scour. Work was performed in conjunction with the New York State Thruway Authority as part of their 1989 special hydraulic reassessment of all Thruway bridges. Thruway bridges were selected for the Ground Penetrating Radar Study based on site observations, water depth, and soil conditions. A summary of ground penetrating radar technology, its uses, and effectiveness are given; geophysical properties and limitations are compared to current dive inspections. Guidelines from the FHWA addressing the problem of bridge scour are presented including recent research in 1989 by the USGS. Similarities and differences due to the diversity of glacial deposits throughout New York State are discussed. Finally, the significance of river scour in bridge design is examined relative to river sediments, initial design costs, and the costs of frequent monitoring programs.

BACKGROUND

Ground Penetrating Radar (GPR) also known as Subsurface Interface Radar (SIR) utilizes impulse radar technology to obtain a continuous, high resolution profile of subsurface layers to depths greater than 100 feet depending upon antenna power, and soil electrical properties. Pulsed electromagnetic energy was first used in 1926 by Hulsenbeck[1] to detect buried objects, the method was primitive compared to today's technology. Advancement of the technology was slow until the late 1960's, when more sophisticated applications were needed. While the lunar investigations impelled some progress, the real push for technological advancements came from the military during the Vietnam War[2]. The U.S. Army developed Combat GPR to help locate enemy mines, tunnels and bunkers. The new technological advancements resulted in the successful differentiation of layered media such as air, water, and varying soil conditions. At this point, many commercial applications for GPR were recognized, and in 1970 Geophysical Survey Systems Incorporated (GSSI) was formed and awarded the first known patent for subsurface radar.

GSSI, of Hudson, New Hampshire expanded rapidly as a pioneer in this field. Since 1970 many commercial applications utilizing GPR technology have developed into successful markets. The firm Zane Yost and Associates, Inc. developed a nondestructive test method to detect cracks in reinforced concrete slabs in 1978[2]. Currently, some DOT offices are exploring this use of GPR as part of their bridge inspection procedures, since GPR technology may provide increasing accuracy, a reduction in inspection time, and a large cost savings.

Recent publications by F.P. Haeni (USGS, Hartford, Connecticut, 1989[5]) suggests limited success in penetrating different media. Haeni detected scour in-filling at three Connecticut River Bridges in Hartford, Connecticut. The GPR results were compared to other geophysical methods including a black and white fathometer, color fathometer, and a tuned transducer. Haeni's GPR conclusions showed good definition of scour holes and interfaces in shallow water, limited success in conductive material, scattered signals in dense, rocky subbottoms, and multiple reflections caused, in part, by interference.

INTRODUCTION

According to the Federal Highway Administration (FHWA), bridge scour is one of the major causes of bridge failures in the U.S. today[3]. With this in mind, the need for inspection techniques to quantitatively define the extent of scour is essential. Before the use of GPR to detect scour in riverbed material is discussed, a background of the scour process will be reviewed. This paper will concentrate on the application of GPR used to detect riverbed scour around bridge abutments and piers.

For a detailed listing of other civil engineering applications using GPR technology, the reader is referred to Morey, 1974[4].

Scour is a continual natural geologic phenomenon; most important, is the rate of scour for different materials because bridge pier failures can occur rapidly. While sedimentary bedrock scour may take years to occur, coarse-grained sand or gravel deposits can erode in a matter of days or even hours if adverse hydraulic conditions exist. It is the fluctuation of the hydraulic conditions that make scour prediction and assessment very difficult.

DEFINITION

Riverbed scour is the result of an increase in the flow conditions of a river, which is proportional to the water velocity times the flow area. Obstructions of the flow path produce an increase in water velocity causing water vortices or "eddies". Eddies are strong currents that produce increased water friction forces acting at all flow bound aries. The low flow boundary is where the most scouring can occur, adjacent to river sediment. The complete process of eroding material is called degradation and aggradation. Generally, degradation is the hydraulic removal of material, and aggradation is the redeposition of the same material. The redeposition of material, usually downstream, occurs when the velocity of the water decreases to a rate that allows suspended material to settle back to the river bottom. The aggradation process is time dependent since larger sand particles will settle out first while silts and clays take longer. Degradation is not limited to suspended materials alone; coarse gravel, cobbles, and even riprap can be pushed or rolled along the river bottom by the hydraulic forces and eroding deeper underlying deposits.

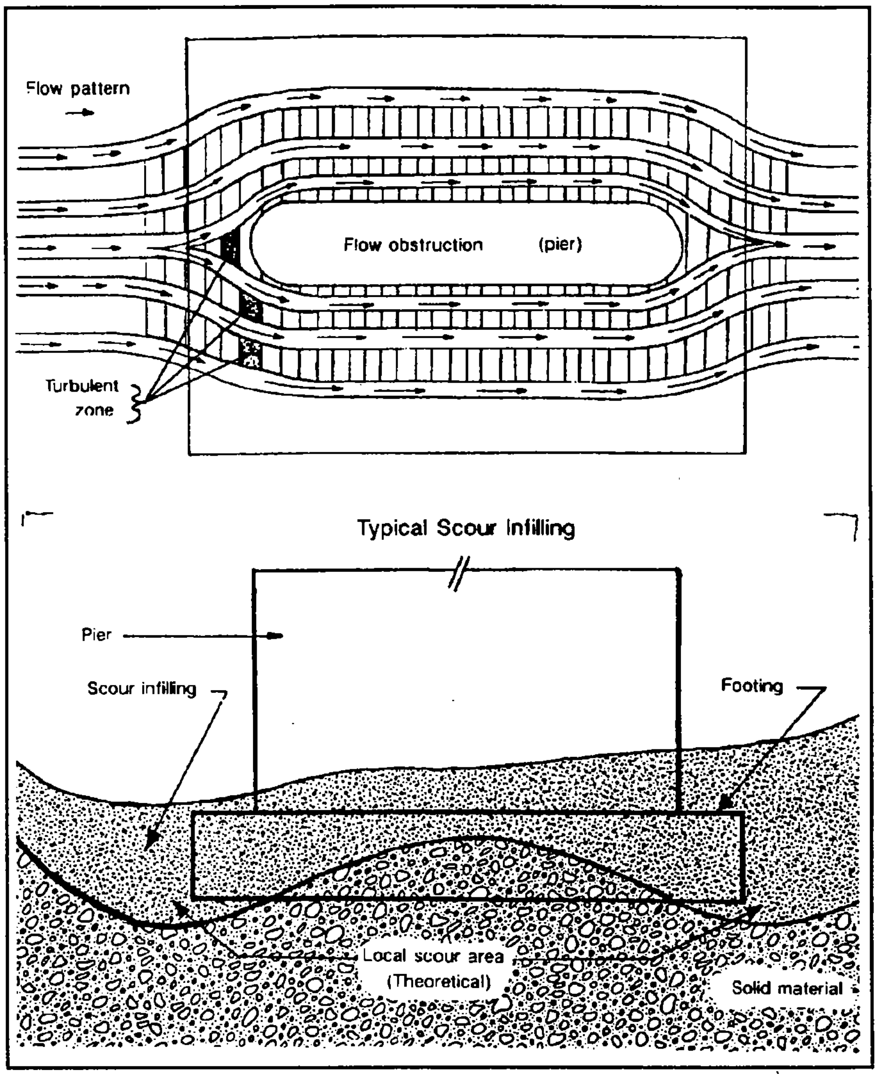

Figure 1: Flow obstruction

TYPES OF SCOUR

Three forms of scour may occur: natural degradation, local scour and contraction scour. The latter two can occur quickly. Local scour is the result of an obstruction of flow which increases the water velocity and produces vortices. These migrating eddies are very difficult to predict and quantify, due to the hydraulic turbulence caused by the obstruction. The (local) area effected is usually close to the obstruction, either adjacent, slightly downstream and most commonly directly upstream of the obstruction. **(See Figure 1)**.

However, when the obstruction affects the entire flow pattern of the water, the term "contraction scour" is used.

Contraction scour is caused both naturally and artificially. Artificial contraction scour is the result of man-made changes in the hydraulic cycle of a river. They include upstream and downstream volume changes caused by flood control measures, bridge placements, and other man-made effects. Examples of natural contraction scour include the meandering features and natural shifting of streams and rivers.

The most serious result is the combination of all three effects. If a bridge crossing is required at an abrupt turn in the stream, the adverse effect of contraction and local scour must be recognized and ac counted for in the design of the structure. Also, as the watershed conditions change during the life of a given structure, the original hydraulic conditions used in the design may no longer be valid requiring further evaluation.

PROBLEM

Scour is a continuous process and can be cumulative. Scour holes may develop during degradation, and after the peak flow is reached, in-filling will occur allowing for aggradation to complete the flood cycle. Subsequent floods will wash out the in-filled material and re start the new erosion, at the point where the previous erosion stopped. With every flood condition, there is a complete degradation and aggradation cycle: in other words, scour in-filling will occur after every flood. It is the scour in-filling that is difficult to detect during the inspection process, since sounding rods are sometimes unsuccessful and it is virtually impossible (and dangerous) to inspect the rate of scour at peak flood conditions. Thus, scour detection during post flood conditions is the goal of any periodic investigation program. Obviously local scour holes are easy to visibly detect if the water depth is shallow. In deeper water, a river bottom profile can be taken with sounding techniques or more advanced methods, such as a fathometer survey. But detecting an in-filled scour hole or the presence of a scour interface is more challenging.

INSPECTION GUIDELINES

Some general guidelines concerning the Federal and State inspection procedures will be discussed in this section. The need for state-of- the-art scour inspection procedures was recognized in the mid 1980's. According to the FHWA[3], in 1985, 73 bridges were destroyed by flood conditions in Pennsylvania, Virginia, and West Virginia; and in the New England area, the 1987 Spring floods damaged 17 bridges. The results of the two floods produced a Technical Advisory (Number 5140.20) from the FHWA, in September 1988. The advisory provided the most recent information available for scour assessment.

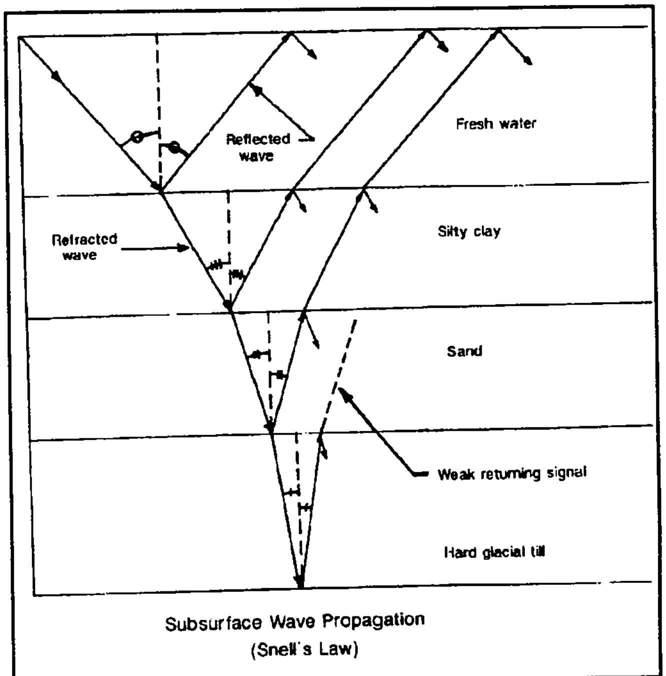

Figure 2: Subsurface wave propagation (Snell's law)

Present guidelines require underwater inspections in rivers that exceed a depth of six feet. The underwater inspections involve scuba divers and video tapes to assess the natural condition of the bridge's substructure[6]. In addition to underwater filming, divers use steel rods to "probe" the subsurface and analyze the consistency of the material. In-filled scour holes, consisting of silts and clays, will show little resistance to the probe, and may easily be washed out. However, the "snowdrift" phenomenon may be deceiving. The flood waters may recede in such a way that the particles settling to the bottom may be "packed together" with some structural integrity, similar to a snowdrift. If the packing occurs around a pier or abutment, the diver's probe may not differentiate the in-filled material from natural deposits. Additionally, the cost of underwater dive inspections have not been mentioned. According to consultants with expertise in this area, costs can range from $5,000 to $10,000 per structure.

In many of these cases, a soil boring may be required using a drill rig mounted on a barge. This process is time consuming and costly, but necessary to fully investigate the subsurface conditions. Usually the number of borings is kept to a minimum, based on an engineering analysis for the specific structure. The borings are usually taken at the front or immediately downstream of the obstruction, where scour is most likely to occur. However, in some cases, the height required for the drill rig's boom makes it difficult to drill near or under the structure due to low clearance. If this occurs, and a scour hole is probable in that location, further inspection is required. The entire inspection process is timely, expensive, and depending on the DOT inspection rating, may be required bi-annually or even annually.

In summary, present traditional inspection techniques are time consuming, expensive, and not always accurate over the entire bridge site. In contrast, GPR technology is faster, less expensive, easily applied to all areas of the stream bed, and can be more accurate in detecting in-filled scour holes. Presently GPR is not a perfect system; however when used in conjunction with the other procedures, GPR can be of significant value in scour detection programs.

CLARKSON'S RESEARCH PROGRAM

Clarkson University was awarded a research grant by the New York State Science and Technology Foundation (NYSSTF) in 1990, to study the effectiveness of GPR in detecting scour holes around bridge foundations. The research period is for one year. Some of the current results are summarized here.

In late 1989, the New York State Thruway Authority (NYSTA) commenced a special investigation program to reassess the hydraulic adequacy of all Thruway bridges crossing rivers and streams. The investigation included hydrological evaluations, structural observations, subsurface exploration, and scour assessment based on the FHWA's recent update. The 559-mile Thruway was divided into three sections; Albany south to New York City, Albany west to Syracuse, and Syracuse west through Buffalo to the Pennsylvania line. The Albany to Syracuse section was awarded to the Consulting Firm of Edwards & Kelcey, Inc., and the soil boring subcontract was awarded to Atlantic Testing Laboratories, Limited (ATL). Clarkson proposed to work in conjunction with NYSTA to gain valuable information on the effectiveness of GPR technology and compare the results to present inspection procedures.

Since GPR is not reliable for all soil conditions, a number of bridge crossings were analyzed to determine locations where the particular GPR equipment would be successful in detecting scour interfaces. Clarkson's equipment consists of a radar unit (Model SR80), data recorder, color monitor, graphic monitor, and a 300 MHz antenna; manufactured by GSSI. According to the manufacturer, the 300 MHz antenna works best in water depths of eight feet or less. Some energy is lost through absorption or conduction of the water. This energy loss, referred to as attenuation, is a major drawback in GPR technology today.

GPR THEORY

An analysis of radar waves and the geophysical properties of different materials is necessary to understand and interpret the GPR survey. The phrase "impulse radar" is frequently used to describe the GPR process. The antenna pulses a wave of electromagnetic energy into the subsurface during the transmit cycle. Upon reaching an interface or buried object, a portion of the energy is reflected back to the surface, while the main pulse continues deeper. The amount of energy that is reflected back to the receiver, and the returning angle depends on the dielectric properties of the material. The process is analogous to Snell's Law of Reflection or refraction of light waves through different media (see **Figure 2**).

Table 1 illustrates the range of dielectric properties for various materials. When an interface or change in material is detected, the direction at which the energy is reflected, and the rate at which it is absorbed can be related to the inherent properties of the material such as the chemical composition, density, and water content. The GPR system measures the energy's two-way travel time from the surface to the interface and back; the depth and thickness of the interface can be calculated knowing the dielectric constants or by ground truth measurements; i.e., soil borings. The depth can be calculated by rearranging the following relationship:

TABLE 1 Dielectric Constants, Conductivities, and Impulse Rates of Various Earth Materials

Approximate Dielectric Constant, ϵ_r	Approximate Conductivity (Energy Loss) $\frac{1}{\text{OHM Meter}}$	Impulse Rate or Penetration Time (Nanoseconds/ft)	Material
1	0	2	Air
81	10^{-4} to 3×10^{-2}	18	Fresh Water
81 to 88	4 to 5	18 to 19	Sea Water
4	10^{-4} to 10^{-2}	4	Fresh Water Ice
8	10^{-9} to 10^{-3}	5.7	Granite
4 to 6	10^{-7} to 10^{-3}	4 to 4.9	Sand, Dry
30	10^{-4} to 10^{-2}	10.9	Sand, Saturated (Fresh Water)
10	10^{-3} to 10^{-2}	6.4	Silt, Saturated (Fresh Water)
8 to 12	10^{-1} to 1	5.7 to 7	Clay, Saturated (Fresh Water)
12	10^{-4} to 10^{-2}	7 to 9	Average Soil

Information contained in Table 1 was obtained from GSSI's Equipment Specifications

$$E_R = (t/_2)^2 \times (c/_d)^2 \qquad \underline{\textbf{EQUATION 1}}$$

where: E_R = relative dielectric permittivty (**Table 1**)
t = two-way travel time in seconds (**Table 1**)
c = speed of light in free space (approx. 10×10^8 ft/sec)
d = depth to the reflected interface (ft)

A finite amount of energy is transmitted from the antenna; portions of the energy are absorbed and reflected back to the surface and eventually the signal dissipates with depth. Hence, the greater the energy source, the greater the depth of penetration. The exception to this rule is in a conductive medium where "attenuation" or the energy absorption increases proportional to the amount of energy pulsed. The large electromagnetic field associated with higher energy pulses can also distort the reflected signal in less conductive material. Advanced models overcame this problem by separating the transmitter and receiver. These higher energy "dual antennas" have been successful in recent years, but there are still difficulties with interference, and some highly conductive materials can not be penetrated; i.e., salt water.

EFFECTIVENESS

Figure 3 illustrates a difficulty associated with GPR inspection of scour holes (recall that local scour occurs very close to the obstruction, such as a pier or an abutment). The antenna's energy footprint is in the shape of an ellipse extending out to the side roughly 30° from the vertical axis. The most accurate information is obtained directly below the antenna, but when the antenna is moved close to an abutment or pier, the signal reflected off the side of the obstruction distorts the returning signal. This is a significant shortcoming since the integrity of the structure is most dependent on the load bearing material adjacent to the foundation.

The effectiveness of GPR is also reduced in fast moving water (with suspended particles), in highly plastic saturated clays, and in dense rocky subbottoms; all of which distort or "scatter" the energy allowing less penetration. Another drawback is the complexity of the system, in general, it is not "user-friendly", which is another main reason that GPR is not widely used for scour inspections today.

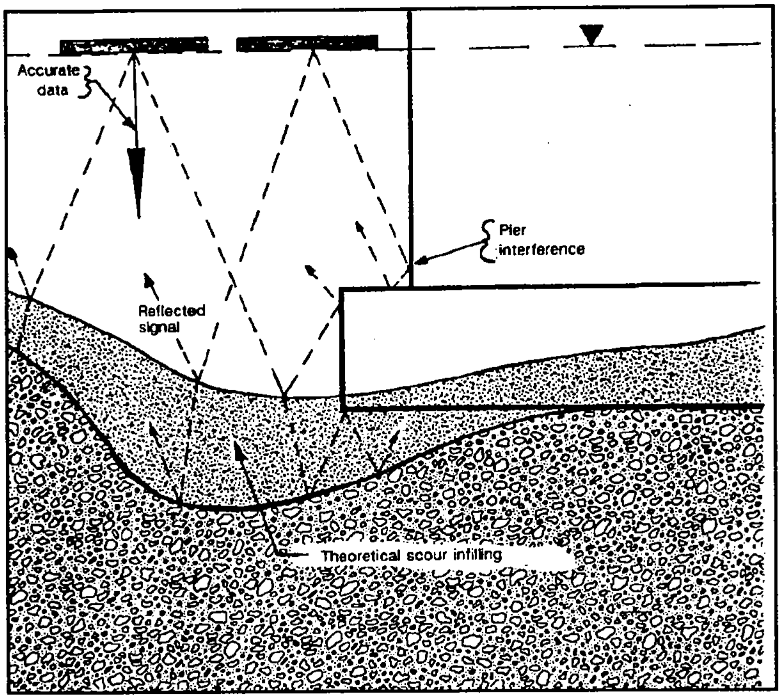

Figure 3: Resultant Pier Interference

monitored the antenna output. The investigation found what appeared to be an interface along the west side of the east pier, recognized by a significant increase in the amplitude of the returning signal. Confirmation was needed and since the GPR investigation took place before ATL was mobilized on Oneida Creek, one additional boring was performed specifically to confirm the scour interface. The split-spoon sampling revealed a sand interface located between two silty-clay layers, at an approximate depth of 4 feet. The material below the sand seam was a stiffer clay, underlain by hard glacial till. Interestingly, irregular eddies were noticed on the water surface above the sand interface. The likely conclusion is that the degradation and aggradation process of a previous flood had deposited the sand in an area where changing hydraulic conditions were evident, and roughly 4 feet of scour in-filling had occurred. While the structural integrity of the spread footing was not affected, radar results did confirm the presence of a scour interface that might not have been detected otherwise.

SITE SELECTION

Equipped with a basic understanding of how the radar system works, the next objective was to successfully find a scour interface. Working in conjunction with the NYSTA, Edwards & Kelcey, and ATL, a selected number of bridge sites were investigated for optimal water depths and soil conditions. Streams with less than 5 feet of water and sandy-silty subbottom sediments were sought; the best site was Oneida Creek located 25 miles east of Syracuse (**Figure 4** shows a cross-section of the site). Only the east pier, supported on a spread footing, experiences constant hydraulic vortices, and as previously mentioned, would be considered the most scour-susceptible part of the structure. A sharp bend in the creek just upstream of the bridge crossing shows natural bank erosion. In 1987, angular stone had been placed around the pier, the additional boring confirmed the presence of this angular stone (**Figure 5**).

RESULTS

The GPR investigation was conducted in the following manner. The antenna was placed in a rubber raft and connected to the radar unit. One person maneuvered the raft around the pier, while the operator recorded and

More importantly, the soil borings provided a ground truth correlation for the soil conditions at that location. Consequently, the other GPR information obtained at that site is considered valid, recorded on a hard copy, and can provide future comparisons without remobilization for the drilling process. Finally, the time required to perform the radar investigation was 10 man hours (five hours in duration with two people). The three soil borings (actual number performed on that bridge) lasted three days, using over 50 man hours, and required a boat, barge, drill rig, crane, traffic control, and various equipment trucks. In contrast, the GPR equipment can be loaded into one van.

Due to the hydrogeology of Central New York State, the Mohawk and Hudson River Valleys consist of rocky-cobbly river sediments where subsurface penetration was ineffective with the current equipment. By contrast, the sand, silt and clay deposits associated with changing water levels in Lake Ontario proved to be a more successful study area for the GPR system. Other rivers, in the Syracuse area, will be investigated using recent soil boring information.

scour data.

ACKNOWLEDGEMENT

The financial support of the NYSSTF to Clarkson University made this investigation possible. All cooperation with work performed in conjunction with the NYSTA, Edwards & Kelcey, and ATL provided valuable data and was greatly appreciated. Clarkson would like to thank Robert Donnaruma and Mark Hixson of the Thruway Authority; Spencer Thew and Thomas Wiggins of ATL; and Frank Huber from Edwards & Kelcey for all their time and effort.

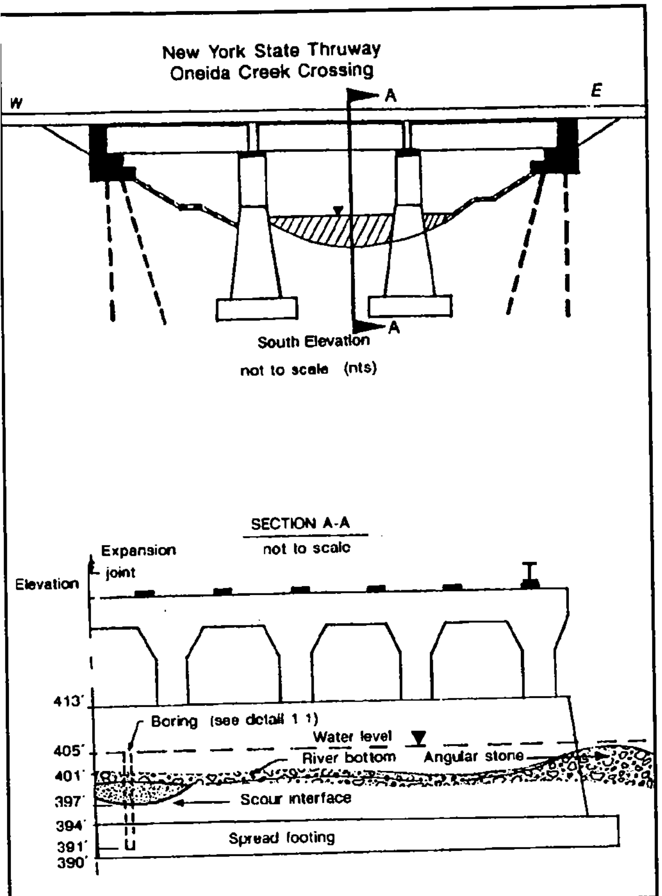

Figure 4: New York State Thruway crossing Oneida Creek.

CONCLUSION

GPR systems range in cost from $20,000 to $60,000; however, cost savings can be justified since State and Federal guidelines require inspections at least every five years. At $5,000 to $25,000 per inspection, the total savings are large, and the savings will increase over the 30 to 40 year lifetime of the bridge. At this time, it appears that continuous scour detection with GPR is not 100% successful. However, certain advantages do exist when GPR is compared to traditional inspection techniques, the most obvious of which is the savings of time and money. Can this savings justify eliminating soil borings and dive inspections altogether? No, not at this time, but it is the author's opinion that GPR used in conjunction with the existing procedures can provide important information essential to future technological advancements of GPR. Finally, the problem of having the process become "user-friendly" can only be solved if future researchers are diligent in their quest for **valid**

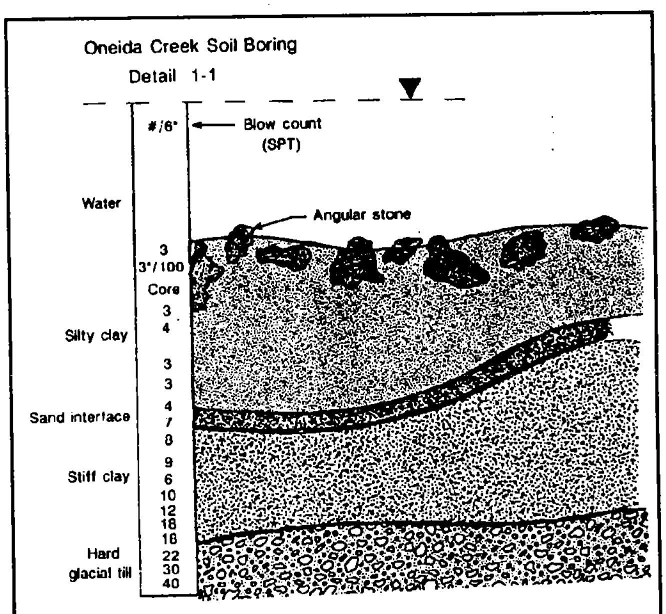

Figure 5: Detail of boring at New York State Thruway crossing Oneida Creek.

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CLUES TO LANDSLIDE IDENTIFICATION AND INVESTIGATION

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INTRODUCTION

The earth surface is constantly being changed by natural forces and activities of man. All earth and rock sloping surfaces are susceptible to landsliding under severe conditions. The geotechnical engineer has the responsibility to evaluate sloping surfaces near transportation facilities to determine the risk of landslide at each site.

The cause and nature of the landslide is usually invisible to us because it is buried deep beneath the surface and may be masked by numerous different geologic deposits and groundwater systems. To accurately predict the nature, shape, and causative factors of a landslide, the investigator must be very thorough. This paper includes a group of clues, guides and rules of thumb assembled to help other investigators.

TYPES OF LANDSLIDES

The nature and type of active landslide must be identified before an evaluation can be made concerning the risk from the slide, or to design corrective treatments. There is an accepted nomenclature for landslides in TRB Special Report 176 "Landslides: Analysis and Control" which will help in communications with others (Ref. 1). The clues assembled here are for rotational slides, translational slides and complex slides. Landslides designated as falls, topples, and flows can be extremely dangerous and it is nearly impossible to predict their occurrence or activity accurately. Therefore, if an area of this type of movement is suspected move the facilities out of harm's way or provide positive protection such as walls or avalanche sheds. These types of slides will not be discussed further here.

MECHANISMS OF LANDSLIDES

The major factors influencing the landslide performance are (1) the slope of the failure plane, (2) the strength on the failure plane, (3) the strength reduction on the failure plane from hydrostatic pressures, and (4) the seepage force in the slope.

Causative Forces

The weight of the soil system is usually known or can be estimated close enough for landslide investigation. Seepage forces from internal groundwater flow are the most difficult items to identify and quantify in landslides, but possible ranges can usually be estimated from surface observations. The water pressure which may build up in any permeable layer or fissure behind the landslide may create an additional and varying force tending to cause failure.

The effects of earthquake loading in landslides may be more difficult to quantify than groundwater, but sufficient background data is available to estimate the degree of risk based on expected earthquake activity.

The major variable which defines the causative force is the angle of the potential sliding surfaces. The steeper the angle, the greater the likelihood of landslide.

Resisting Forces

The primary resistance to landslide movement is the shearing resistance along the failure surface. The shearing resistance along the failure is related to the drained friction angle of the soil on the failure plane. Many landslides have been directly caused by a thin, nearly indetectable, layer of clay (with drained friction angles from 20-25 degrees) found in an otherwise strong material with drained friction angles of 35-45 degrees.
The resistance to failure can also be dramatically reduced by the introduction of water pressure on the failure surface which has the effect of reducing the frictional resistance by reducing the effective normal force on the failure plane.

LANDSLIDE INVESTIGATION GUIDES

Philosophy - Landslide investigations are costly, so carry out preliminary analyses to look for possible controlling features of landslide before starting a field investigation program.

A drill hole may cost over $5,000 to obtain and often does not yield all the information needed. Hundreds of different analyses can be run for that cost. Use the analyses to help design an exploration program by looking for features that the analysis indicates could control the slide.

One way to help identify the accuracy of field information and the selected failure model is to run mathematical analyses assigning limiting values to the variables.

- First test the model by using drained friction angles at upper and lower bounds of the material that was encountered in the preliminary inspection. Then test the analysis using parameters for the worst soil found in the general area of the slide.

- If there are springs outletting on the slope, assume that there are no artesian pressures in the system.

- Use the information from the subsurface exploration program to estimate limiting upper and lower bounds for potential failure slope angles and possibly limiting bounds on the drained friction angle.

- Exploration programs seldom (if ever) can identify a very thin layer with confidence. Therefore, the lower bound of drained friction angle must sometimes be used. In New York State, the lower bound value of drained friction angle is usually about 20 degrees.

The investigation should be a continual reiterative process repeated for each new piece of information obtained. The confidence set in final predictions depends on the degree of certainty that a piece of information is correct, and if that information controls the actions of the landslide. Any assumptions made in the process must be proven or accounted for in the final evaluation.

<u>Low Priority Efforts</u>

1. General ground survey or existing USGS maps are used to identify the overall surface topography of the slide area. Details of topography may trigger a slide, but seldom control a slide.

2. The weight of soil can be closely estimated by using geologic mapping information and possibly a few borings or test pits. The weight of soil seldom controls slides.

3. Groundwater level varies with rainfall and seasons and is, therefore, nearly impossible to accurately quantify. However, it can be assumed that the natural groundwater table is less than five feet below the surface in most fine grained natural slopes in the northeast.

<u>High Priority Efforts</u>

1. Subsurface Profile

The angle of the failure plane is extremely important to the landslide evaluation process. Limitations in exploration technology sometimes negate our ability to positively define the controlling subsurface conditions. Therefore, the profile information should be tested for the most likely failure model including all other information available. Examples of surprises are: a bentonite seam between competent rock layers, or a water bearing gravel seam in an otherwise extremely dense impermeable clay.

A general guide is to look for the obvious first and to look for the exception only when the information used in your mathematical model does not produce results consistent with the obvious.

2. Drained Friction Angle

The drained friction angle controlling the failure strengths in the landslide are usually almost directly related to the plasticity of the fine portion of the material and, therefore, can be estimated quite accurately if you can obtain the plasticity index of the material controlling the failure. (Ref. 2 and Figure 1). For higher P.I. clays, refer to a broad band curve in Bjerrum paper (Reference 3).

3. Groundwater

The controlling hydrostatic pressure on the failure plane is very difficult to quantify. Often, the soil system surrounding the failure plane is nearly impermeable and, therefore, it becomes difficult to obtain accurate

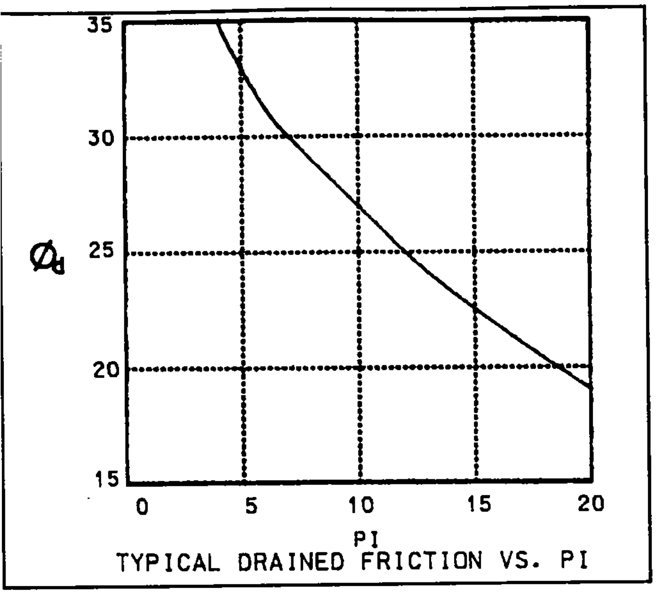


Figure 1

groundwater measurements without extraordinary exploration techniques. If the exact location of the failure plane is not clearly defined from other data, it is difficult to put in a sufficient number of observation wells to identify the critical water pressure system controlling the landslide.

The water pressure in a landslide may vary hourly, daily, weekly, monthly or yearly depending upon the nature of the soil system (Figure 2). Therefore, it is difficult to obtain records during the most critical time to fully quantify the controlling hydrostatic pressure on the failure surface. Other clues such as rainfall vs. movement records are needed to quantify maximum hydrostatic pressures (Figure 3).

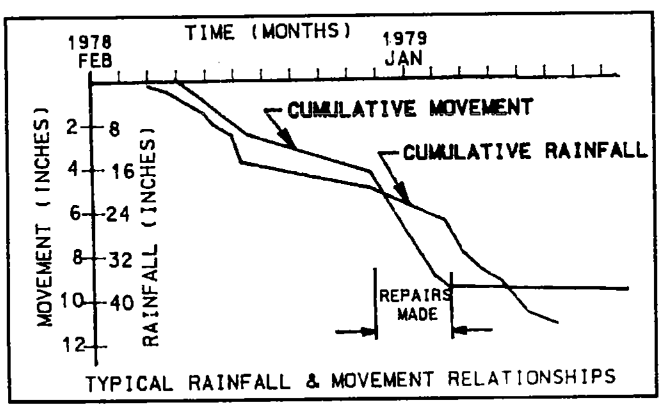


Figure 2

4. Movement Records

Include measurements of landslide movement rates and directions in the landslide investigation. Rainfall and groundwater records are useful to compare with the movement records (Figure 3). The physical slide and the mathematical model selected must respond in the same manner to changes in rainfall or groundwater that have been measured or there is improper variable within the model.

Movement meaningful to landslide investigations can be obtained directly by survey or field instrumentation (slope indicators etc.) if sequential measurements are taken at a significant location of the landslide. Sometimes the amount of blacktop patching used per year or the amount of railroad track cut out of a bend are adequate to reflect movement relateed to time (and therefore rainfall-from meteralogical records at airports etc.).

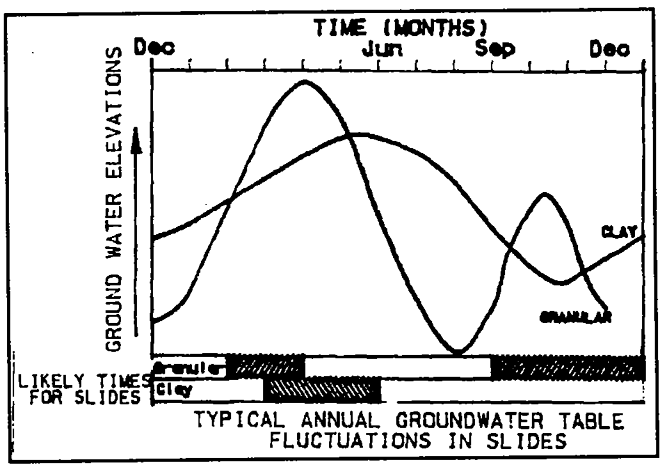


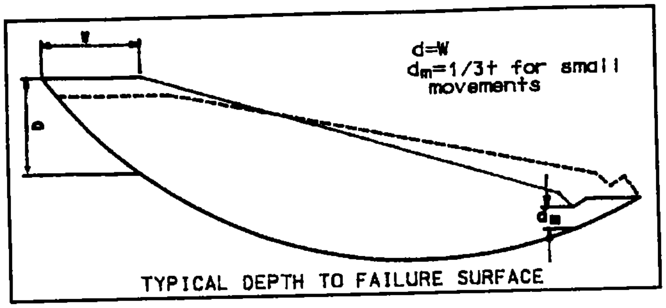
Figure 3

A FEW USEFUL CLUES

1. Depth of Failure Surface

The depth of the failure surface below the crest of the slope is usually equal to the distance from the crest of the slope back to the furthest shear crack. (The depth is usually equal to the distance from the break in slope by the guide rail to the further shear crack in the middle of the pavement). (Figure 4).

For failures beyond the toe of slope, the depth of the failure plane at the toe is usually about one-third of the



TYPICAL DEPTH TO FAILURE SURFACE

Figure 4

distance from the toe to the edge of the mud wave. (Figure 4).

If the mud wave exits on a continuing slope, the outlet of the failure surface is usually near the top of the visible mud wave.

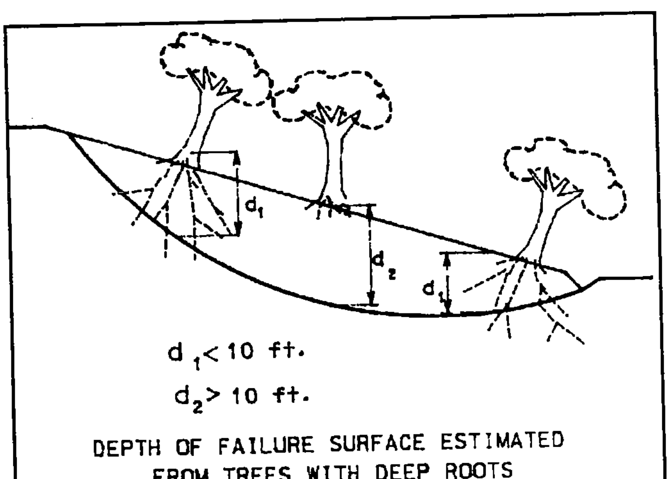
The depth of the failure plane is less than 10 feet if the deep rooted trees in the sliding mass tend to tilt downslope. (Figure 5).

The slope of a buried rock surface can usually be estimated from the slope of the rock surface in geologically similar adjacent areas when orientation of slope to bedding is similar.

Breaks in buried utilities, such as culverts and sewer pipes can give a direct visual identification of where the failure surface exists and sometimes how much movement has occurred across the failure surface.

2. Limits of the Landslide

The exact limits of the landslide can seldom be identified unless very large movements (over two foot vertical drop) have occurred.



DEPTH OF FAILURE SURFACE ESTIMATED FROM TREES WITH DEEP ROOTS

Figure 5

Utilities often are the first to exhibit signs of movement. Telephone poles tend to tilt in the slide and often will cause greater tension or create sagging in the wires between poles (Figure 6). Therefore, the limit of a slide can often be identified based on the amount of tension in the wires compared to the average tension between adjacent poles.

The lateral limits of the slide can often be detected by tears in the grass sod. When tears are not evident, sometimes separations between the soil and roots on major trees are observable.

Old manmade features were usually built neat, in specific geometric shapes (straight line or curve) and, therefore, deviations from the pattern in walls, guide rails, driveways or sidewalks may be evidence that there has been a differential ground movement.

The toe of a slide often shows up in stream. Therefore, extra care is needed in trying to identify the failure limits when they are below water and not directly visible at all times of the year. When a stream is undercutting the toe of a landslide, the mud wave that results usually creates a reverse banana shape island in the stream. (Figure 7). The currents will show different surface wave patterns over the island even though it may not be visible. Also, changes in the normal stream flow patterns are often the results of a mud wave shoving the stream to a different position.

By comparing the normal tree growth pattern observed on an unfailed slope to the pattern which the landslide area you can get an indication of the overall limits of the slide.

It is common for surface drainage to change direction near the edges of a slide because the slide has loosened the normal protective vegetation or stone cover allowing accelerated erosion.

3. Important Landslide Investigation Resources

The landslide investigation process is always unique and sites specific, but here are some suggestions to help in most landslide investigation.

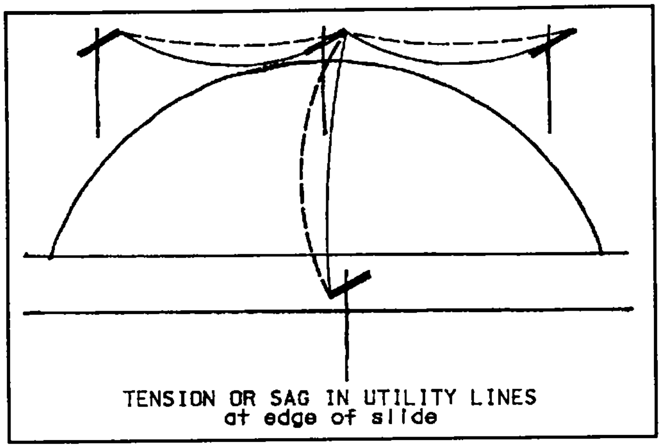


Figure 6

• U.S. Geologic Survey topographic maps of most areas of the United States are available and should be used as the starting point for landslide investigation. These maps will allow a quick evaluation of the adjacent land forms and topography which might influence how to investigate the specific site. The maps can also indicate primary and secondary drainage patterns and sometimes will help identify soil and rock deposits based on land forms.

• Aerial photographs from various planning, agricultural or military groups have been taken at various times starting just prior to World War II and continuing to the present and are available over much of the United States. These photos often do not meet our present day rigorous standards for accuracy, but characteristic shapes of old landslides are easily detectable even in the poorest of photos. Differences of landforms and drainage are also readily visible in most photos as are manmade activities and changes in stream flow or erosion patterns. Springs and rock outcrops are also quite evident. With care, the amount of movement that roadways or other manmade features undergo in landslides from year to year may be quantified from photos taken in different years.

• Geologic evaluations of the immediate and adjacent terrain are essential to an understanding of a landslide. The geologic evaluations can usually identify the history of the rock formations, the glacial formations, and the post-glacial formations. The geologic study can also help to identify recent changes in topography that are now masking critical features. Therefore, a good geologic evaluation of the general area is essential before planning an exploration program.

4. Landslide Modeling Hints

• Mathematical modeling of alternative hypotheses is essential to a complete landslide investigation. Since the factors controlling the performance of the landslide can seldom be positively identified, data gathered must be repeatedly tested against the total system hypotheses. Hundreds of mathematical models can be evaluated at a cost much less than one subsurface exploration or test pit.

• Numerous alternative solutions need to be modeled early in the investigation process. Some corrective treatments will work on all possible models and, therefore, no detailed investigation process is needed. Conversely, if the desired corrective treatments don't work for all reasonable models, then an intensive investigation is needed to prove or disprove the existence of the critical factors in the models that would cause failure. (For example: a toe wall is desirable to stop a slide. A thin clay layer under the wall would result in continued movement after wall construction. Then you must prove the clay doesn't exist (test pits), design a shear key under the wall, or choose another solution). The following decision points should be clarified to direct the proper level of investigations:

• Are lives endangered?

• Is this a known slide?

• Is the goal to predict performance (close road)?

• Is the goal to stop a slide?

• Is the goal to predict if a slide could occur?

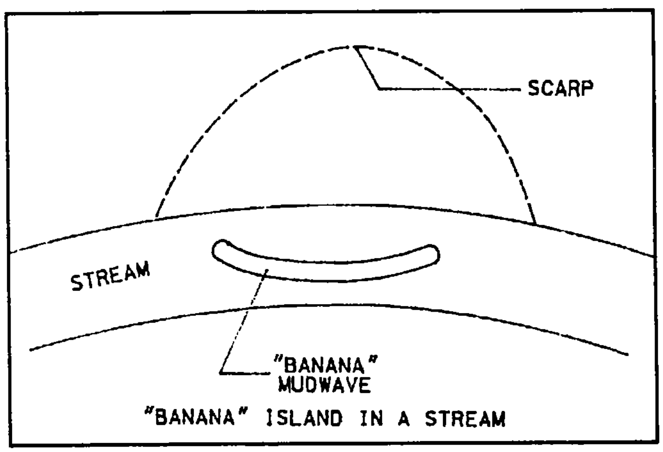


Figure 7

- When using New York State DOT procedures the amount and rate of movement can be related to the Factor of Safety (FS) before failure. (Figure 8).

- Small movement (inches) usually means the FS before failure varied from 0.95 to 1.1.

- Moderate movements (feet) usually means the FS before failure varied from 0.9 to 1.0.

- Large movements (including liquefaction) usually mean the FS before failure was below 0.9. A FS below 0.6 has no meaning (usually an incorrect model).

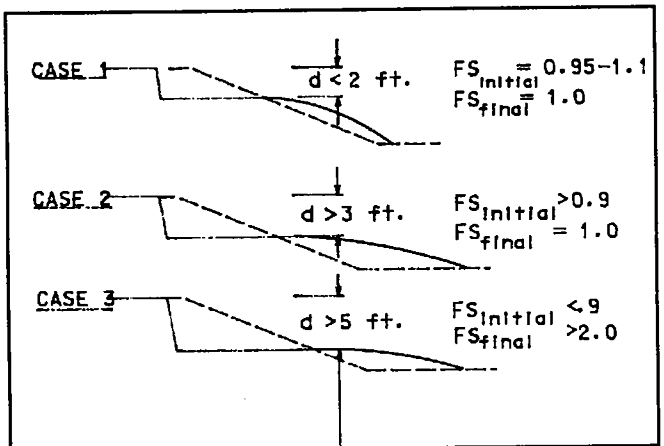


Figure 8: Typical factors of safety (FS) for average 40-50 foot high slope.

SUMMARY

The investigation processes for landslides must be site specific, but much wasted effort can be saved by following an ordered process of analysis and field investigation. Asking good questions and reading the clues that mother nature has provided will save much effort. Start with the simplest, most obvious model before investigating more complex models. Remember to go back and compare the final answers against the simplest model.

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SLOPE FAILURE PROBABILITY FOR MIXED LAYERED SOILS

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ABSTRACT

Because highway personnel are more interested in whether a slope will be stable or fail, the probability of failure is a better measure of stability than the factor of safety. Probability of failure considers the number of tests used to find the soil strength and the variation of strength within a layer.

This paper describes a statistical method to evaluate the stability of an earth slope. The statistical method is based on the "Point Estimate" method and allows computation of a probability of failure in slopes which contain soils with cohesion and internal friction. An example problem is contained in the paper.

The method can be summarized by the following steps:

1. Find the mean and standard deviation of the soil strength (cohesion and internal friction).
2. Find the correlation coefficient between the cohesion and internal friction for each layer.
3. Find the factors of safety for the slope using combinations of high and low soil strength values.
4. Find the weighing functions.
5. Find the expected value and standard deviation of the factors of safety.
6. Find the probability of failure from a normal distribution table.

Safer and more economical highway slopes can be designed using the method.

INTRODUCTION

In order to check the stability of a highway slope, a "factor of safety" (FS) is traditionally calculated. Highway personnel, however, are more interested in whether the slope will fail or be stable. As a result, the "probability of failure" is a better measure of stability than "factor of safety".

High probabilities of failure can occur when the soil strength is variable or based on only a few tests. Soil variability can be compensated for by using "engineering judgement" and increasing the required FS. Knowing the probability of failure improves engineering judgement by providing a rational basis for making a safe and economical highway slope design.

An example of the probability of failure was presented at the 1988 Highway Geology Symposium by J. R. Verduin and C. W. Lovell. In the example, a slope containing one soil, the expected FS is 1.413. However, the probability that the FS is below 1.0, where failure is assumed to occur, is 2.87%.

In a paper at the Highway Geology Symposium last year (Garrett and Thornton, 1990), the method was extended to include more than one layer of soil where each layer contained either cohesion or internal friction (but not both). The example presented contained four soil layers and had an expected FS of 1.259. The probability that the FS is less than 1.0, however, is only 0.62%.

Comparing the two examples, the slope with the highest FS is over four times as likely to fail! These surprising results are caused by the number of soil strength tests and variation of the strength within each soil layer.

Applications of the probability of failure method were presented for one soil with cohesion and internal friction (Verduin and Lovell, 1988) and for layers of soil with either cohesion or internal friction (Garrett and Thornton, 1990; McGuffey, Grivas, Iori, and Kyfor, 1982). These papers describes the method for two layers of soil with cohesion and internal friction.).

BACKGROUND

The probability of failure method is based on the "Point Estimate Method" (PEM) which was developed by Rosenblueth (1975 and 1981) and described by Harr (1987). In the PEM, a distribution of the variable must be found or assumed. If a normal distribution is assumed, the problem is simplified. Details of the PEM development and a discussion of other distributions are contained in a thesis by Garrett (1989), and a paper by McGuffey, Iori, Kyfor, and Grivas (1981).

The basic equations necessary to solve a slope stability problem by finding the probability of failure are given below.

Basic Statistics:

mean: $$\bar{X} = \frac{X_1+X_2+X_3.....+X^n}{n}$$ equation 1

where: $X_1, X_2, X_3, ... X_n$ are values in a set of data (strength tests)

n is the total number of values

estimate of variance: $$\sigma^2 = \frac{\Sigma(Xi-\bar{X})^2}{n-1}$$ equation 2

The variance is a measure of the scatter of a random variable about its mean.

estimate of standard deviation: $$\sigma = \left[\frac{\Sigma(Xi-X)^2}{(n-1)}\right]^{.5}$$ equation 3

The standard deviation (σ) is a useful measure of the dispersion of a random variable about its mean.

correlation coefficient, r

$$r = \frac{N\Sigma XY - \Sigma X \Sigma Y}{\sqrt{[N\Sigma X^2-(\Sigma X)^2]\,[N\Sigma Y^2-(\Sigma Y)^2]}}$$ equation 4

The correlation coefficient is based on a straight line fit between two dependent variables. Cohesion and internal friction are the variables of concern in the slope stability problem. If there is a perfect correlation, i.e. the value of the cohesion is directly related to the value of internal friction, the correlation coefficient will be +1.0 (if they both increase) or -1.0 (if one decreases when the other increases).

Point Estimate Method For Slopes:

Values for strength a standard deviation away from the mean are called strength plus and strength minus values.

For cohesion:

$$\text{cohesion plus} = C+ = \bar{C} + \sigma(C) \qquad \text{equation 5}$$

$$\text{cohesion minus} = C- = \bar{C} - \sigma(C) \qquad \text{equation 6}$$

where $\bar{C}$ is the mean cohesion value

$\bar{\sigma}$ is the standard deviation of the cohesion in that layer

For internal friction:

$$\text{phi Plus} = \phi+ = \bar{\phi} + \sigma(\phi) \qquad \text{equation 7}$$

$$\text{phi minus} = \phi- = \bar{\phi} - \sigma(\phi) \qquad \text{equation 8}$$

where $\bar{\phi}$ is the mean internal friction

$\sigma(\phi)$ is the standard deviation of internal friction

The FS is a function of the strength of each layer [FS = f(c, ϕ_1, C_2, ϕ_2)]. Therefore, a FS must be found for each combination of soil strength C+, C-, ϕ+ and ϕ-. The number of FS's to be found then, is 2^n where n is the number of strength variables.

A slope stability problem with two layers, each with C and ϕ, would require 16 FS's. If one layer had only a C or ϕ, 8 FS's would be required.

The symbol FS++++ is used for the FS for a two layer slope with C+ and ϕ+ used for strength values in both layers. FS-+++ is the symbol for the FS when C- and ϕ+ are used for the first layer and C+ and ϕ+ are used for the second layer.

Normal Distribution:

If the assumption is made that there is no correlation between layers (i.e. the soil strength in one layer is independent of the strength in another layer), the points which approximate the joint distribution of the random variable are:

$$p_{++++} = p_{----} = p_{++--} = p_{--++} = \frac{1}{16}(1 + r_{c1\phi1} + r_{c2\phi2}) \qquad \text{equation 9a}$$

$$p_{+++-} = p_{---+} = p_{++-+} = p_{--+-} = \frac{1}{16}(1 + r_{c1\phi1} - r_{c2\phi2}) \qquad \text{equation 9b}$$

$$p_{+---} = p_{-+++} = p_{+-++} = p_{-+--} = \frac{1}{16}(1 - r_{c1\phi1} + r_{c2\phi2}) \qquad \text{equation 9c}$$

$$p_{+-+-} = p_{-+-+} = p_{+--+} = p_{-++-} = \frac{1}{16}(1 - r_{c1\phi1} - r_{c2\phi2}) \qquad \text{equation 9d}$$

The expected FS, E[FS], then is:

$$\begin{aligned} E[FS] = \; & p_{++++}FS_{++++} + p_{+++-}FS_{+++-} + p_{++--}FS_{++--} + \\ & p_{+---}FS_{+---} + p_{----}FS_{----} + p_{---+}FS_{---+} + \\ & p_{--++}FS_{--++} + p_{-+++}FS_{-+++} + p_{-+-+}FS_{-+-+} + \\ & p_{-++-}FS_{-++-} + p_{+--+}FS_{+--+} + p_{++-+}FS_{++-+} + \\ & p_{--+-}FS_{--+-} + p_{+-+-}FS_{+-+-} + p_{+-++}FS_{+-++} + \\ & p_{-+--}FS_{-+--} \end{aligned} \qquad \text{equation 10}$$

And the expected value of the squared FS's, $E[FS^2]$, is:

$$\begin{aligned} E[FS^2] = \; & p_{++++}(FS_{++++})^2 + p_{+++-}(FS_{+++-})^2 + \\ & p_{++--}(FS_{++--})^2 + p_{+---}(FS_{+---})^2 + \\ & p_{----}(FS_{----})^2 + p_{---+}(FS_{---+})^2 + \\ & p_{--++}(FS_{--++})^2 + p_{-+++}(FS_{-+++})^2 + \\ & p_{-+-+}(FS_{-+-+})^2 + p_{-++-}(FS_{-++-})^2 + \\ & p_{+--+}(FS_{+--+})^2 + p_{++-+}(FS_{++-+})^2 + \\ & p_{--+-}(FS_{--+-})^2 + p_{+-+-}(FS_{+-+-})^2 + \\ & p_{+-++}(FS_{+-++})^2 + p_{-+--}(FS_{-+--})^2 \end{aligned} \qquad \text{equation 11}$$

The standard deviation of the FS's, σ [FS], is:

$$\sigma[FS] = \sqrt{E[FS^2] - E[FS]^2} \qquad \text{equation 12}$$

For a normal distribution, the standardized variable, Z, is:

$$Z = \frac{FS-E[FS]}{\sigma[FS]} \qquad \text{equation 13}$$

where FS is the cutoff value of the normal distribution table (e.g. FS = 1.0)

From the standardized variable, Z, a normal distribution table (Table 1) can be used to find the probability that a value will be less than Z.

Table 1: Normal Distribution Table

Z	0	-.5	-1.0	-1.5	-2.0	-2.5	-3.0
Probability	.5	.3085	.1587	.0668	.0228	.0062	.0013

Example

Consider the slope in Figure 1. The slope consists of two layers of soil both with c and ϕ. In order to determine the factor of safety against failure, samples were gathered and tested in the laboratory to determine their strength parameters, c and ϕ. The strength test results are given in Table 2.

An engineer might assume that the two soil layers in Table 2 are the same since the numbers are close and both appear to be a silt. For this example, however, the layers are assumed to be different to illustrate the most difficult case.

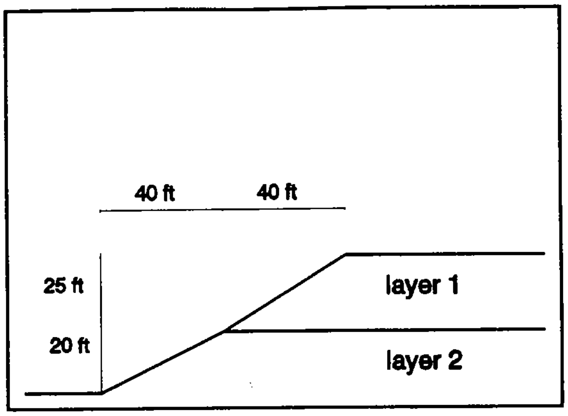

Figure 1
Example of a Two Layer Slope

Table 2
Results of Laboratory Test on Samples

Unit Weight Layer 1 = 110 pcf
Unit Weight Layer 2 = 120 pcf

test	First Layer c(pfs)	First Layer $\phi°$	Second Layer c(pfs)	Second Layer $\phi°$
1	200	31	150	27
2	180	33	110	30
3	210	28	240	24
4	230	27	220	25
5	160	34	120	32
mean	196	30.6	168	27.6
σ	27	3.05	58.9	3.36

The high and low values for cohesion and internal friction are:

$C_1+ = 223$ psf $\phi_1+ = 33.65°$
$C_1- = 169$ psf $\phi_1- = 27.55°$
$C_2+ = 226.9$ psf $\phi_2+ = 30.96°$
$C_2- = 109.1$ psf $\phi_2- = 24.24°$

The next step is to find the correlation coefficients between c_1, ϕ_1 and C_2, ϕ_2 from Equation 4. The correlation coefficients are found to be -0.964 for c_1, ϕ_1 and -0.927 for c_2, ϕ_2.

Next the points approximating the joint distribution of the random variables can be found from Equation 9 (a,b,c,d):

$$p_{++++} = p_{----} = p_{++--} = p_{--++} = (1-.964-.927)/16 = -.05569$$
$$p_{+++-} = p_{---+} = p_{++-+} = p_{--+-} = (1-.964+.927)/16 = 0.0602$$
$$p_{+---} = p_{-+++} = p_{+-++} = p_{-+--} = (1+.964-.927)/16 = 0.0648$$
$$p_{+-+-} = p_{-+-+} = p_{+--+} = p_{-++-} = (1+.964+.927)/16 = 0.1807$$

A slope stability analysis (Simplified Bishop Method) was performed on different combinations of c and ϕ to determine to factors of safety against failure. In this case, the slope stability program was run on the different combinations of high and low values for c and ϕ. The values for c and ϕ along with the corresponding values for FS are given in Table 3.

Table 3
High and Low Combinations and Factors of Safety

	c_1(psf)	$\phi_1°$	c_2(psf)	$\phi_2°$	FS
----	169	27.55	109.1	24.24	1.1413
---+	169	27.55	109.1	30.96	1.3573
--++	169	27.55	226.9	30.96	1.5226
-+++	169	33.65	226.9	30.96	1.5897
-+-+	223	33.64	109.1	30.96	1.4579
+-+-	223	27.55	226.9	24.24	1.3295
+--+	223	27.55	109.1	30.96	1.3977
-++-	169	33.65	226.9	24.24	1.3527
++-+	223	33.65	109.1	30.96	1.4560
--+-	169	27.55	226.9	24.24	1.3014
+-++	223	27.55	226.9	30.96	1.5595
-+--	169	33.65	109.1	24.24	1.1873

From the points and the factors of safety the expected value and standard deviation of the factor of safety against slope failure can be determined. Solving Equations 10, 11, 12, the expected value and standard deviation are:

$$
\begin{aligned}
E[FS] = \; & -.05569(1.6235) + 0.0602(1.333798) + -.05569(1.2123) \\
& + 0.0648(1.1714) + -.05569(1.1413) + 0.0602(1.3573) \\
& + -.05569(1.5226) + 0.0648(1.5897) \\
& + 0.1807(1.4579) + 0.1807(1.3527) + 0.1807(1.3977) \\
& + 0.0602(1.4560 +).0602(1.3014) + 0.1807\,(1.3295) \\
& + 0.0648(1.5595) + 0.0648(1.1873) \\
= \; & 1.3821
\end{aligned}
$$

$$
\begin{aligned}
E[FS^2] = \; & -.05569(1.6235)^2 + 0.0602(1.3798)^2 + \\
& -.05569(1.2123)^2 + 0.0648(1.1714)^2 + \\
& -.05569(1.1413)^2 + 0.0602(1.3573)^2 + \\
& -.05569(1.5226)^2 + 0.0648(1.5897)^2 + \\
& 0.1807(1.4578)^2 + 0.1807(1.3527)^2 + \\
& 0.1807(1.3977)^2 + 0.0602(1.4560)^2 + \\
& 0.0602(1.3014)^2 + 0.1807(1.3295)^2 + \\
& 0.0648(1.5595)^2 + 0.0648(1.1873)^2 \\
= \; & 1.9136
\end{aligned}
$$

$$\sigma[FS] = \sqrt{1.9136 - 1.3821^2} = 0.05798$$

To find the probability that the FS is less than 1.3, the Z from equation 13 is found

$$Z = \frac{1.3 - 1.3821}{0.05798} = -1.416$$

Then from Table 1, the probability associated with Z is .08 or 8%. Therefore, there is an eight percent probability that the FS is less than 1.3. The probability that the FS is less than 1.0 is less than .02%.

CONCLUSION

The slope failure probability methods provides useful information to design safe economical highway slopes.

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NORTHERN NEW ENGLAND LANDSLIDES

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ABSTRACT

Northern New England landslides, and particularly those in Vermont, have been studied in some detail. These studies, conducted under a joint U.S. Geological Survey-Vermont Geological Survey project, extended from 1983 to 1990. The purpose of this project was to determine the geological processes causing landslides.

While many hundreds of both soil and rock landslides have occurred in mountainous terrain, lower elevations are not immune. Evidence of creep, as well as failures, have been observed on many less steep slopes.

The length and width of many soil landslides, such as debris slides and earth flows, were measured. Auger holes were made on several of these landslides to measure thicknesses of the failed masses and obtain samples for laboratory analysis. The measurements were used to determine the volume of material that had moved and, where possible, the location of the failure surfaces.

Grain size determinations and shear strength tests were made on numerous landslide materials. Many of the latter tests were made in the field using the Torvane and Penetrometer. Short Shelby Tube samples were obtained from some landslide substrate materials for laboratory unconfined compressive strength tests. These data were used to determine the physical conditions necessary for landsliding in the particular material under study.

Hydrometer testing and X-ray diffractometer analyses were performed on the soil fraction passing the 200 mesh screen. These tests indicated that the bulk of the silt-clay fraction in soil slides consists of silt-sized quartz rather than clay minerals.

Most landslides in the northern New England region occur from late spring through early summer during and after snowmelt and occasionally, late fall. They are usually accompanied by a minimum extreme heavy precipitation of 8 cm or more per day. Wild streams generated by such storms, in addition to flooding, trigger numerous slope failures. The rockfalls that occurred in the Green and Taconic Mountains and in large highway rock cuts were of singular interest. These rockfalls took place on clear days, a day or two after an intense, early summer rain storm.

ACKNOWLEDGMENTS

We wish to thank Dr. Charles A. Ratte for his support of this project and his providing us with the photographs and other data related to the Fairlee Palisades rockfall.

INTRODUCTION

Landslides in northern New England involve both rock and surficial soils. The rock types vary from massive granites and quartzites to thin-bedded phyllites. Surficial deposits subject to sliding range from glacial lakebed deposits to dense basal tills.

Most of the rock failures studied in northern New England have been aided by many seasons of freeze-thaw. Massive rock, such as the Cambrian Underhill and Hoosac Formations, are found at higher elevations in the Mount Mansfield and the Smugglers Notch area of the Green Mountains (fig. 1). Very large joint-bounded blocks weighing thousands of tons have failed in this location (Baskerville and others, 1988). Failures of the Mount Mansfield blocks have been either single rockfalls or they have triggered debris slides in their downward travel upon reaching tree line.

In less massive rock, such as the phyllitic schists on

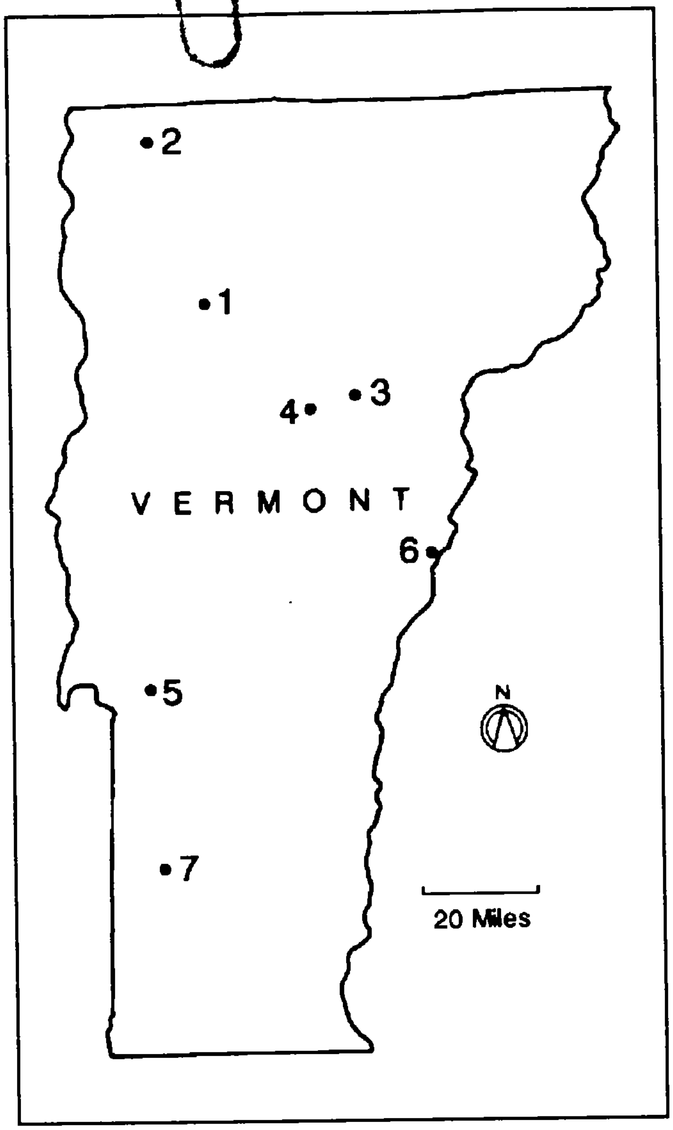

Figure 1: Location map showing sites described in the text: 1-Mt. Mansfield-Smugglers Notch, 2-Enosberg Falls, 3-Plainfield, 4-Montpelier-Gould Hill Rd., 5-Rutland-Castleton, 6-Thetford-Fairlee, 7-Dorset Mountain.

Dorset Mountain in the Taconic Range, landsliding may begin as debris avalanches (Ratte and Rhodes, 1981). A further example of movement in non-massive rocks is creep. Creep has been observed in the slates and phyllites of the Devonian Littleton Formation in outcrops formed by cuts on Interstate 91.

Numerous landslides in surficial materials were observed. Some were studied in detail, along highways, river banks, and on mountain slopes. A landslide in 1990 on the south bank of the Missisquoi River in probable Glacial Lake Vermont deposits (Doll and others, 1970) west of Enosberg Falls (fig. 1) dammed the river. The river quickly created a new meander, detouring the stream around the slide toe.

Similar river bank slides triggered by undercutting on meander bends occurred after extremely heavy rains on tributaries to the Winooski River. Examples of this are to be found along the Great Brook in Plainfield and the North Branch in Montpelier (fig. 1).

Heavy basal till, composed of a hard gray silty-clay when dry, becomes gummy and slippery when wet. A slide in this material occurred in the Castleton River Valley between West Rutland and Castleton (fig. 1). The Castleton River slide developed on the north-facing slope of Scenic Route 4A which is aligned along a sidehill cut through the valley. The toe of the landslide ousted the Castleton River from its channel, flooding adjacent fields. This slide has shown evidence of movement from time to time throughout the 1980's.

The makeup and characteristics of a few examples of both soil and rock landslides are described here to give the reader a flavor for Northern Appalachian landslides. Among these are the soil failures in the Castleton River Valley and the Great Brook and North Branch tributaries in the Winooski River Basin mentioned above. The Fairlee rock slide on I-91, and a debris slide on Dorset Mountain are included as examples of landslides in rock.

LANDSLIDES IN SURFICIAL MATERIALS

In the study of landslides in northern New England, all soil types have been involved in sliding at sometime. These soils are inorganic, ranging from fine- to coarse-grained (ASTM, 1989).

The least porous soils in the group are dense basal tills containing high percentages of clay-sized impermeable material in the intergranular matrix. The most porous soils are very coarse well-rounded boulder tills practically void of fines. Examples of movement in a basal till and landslides involving several soil types from clean tills and alluvium to rhythmites will be examined. These materials typify landslides in surficial materials.

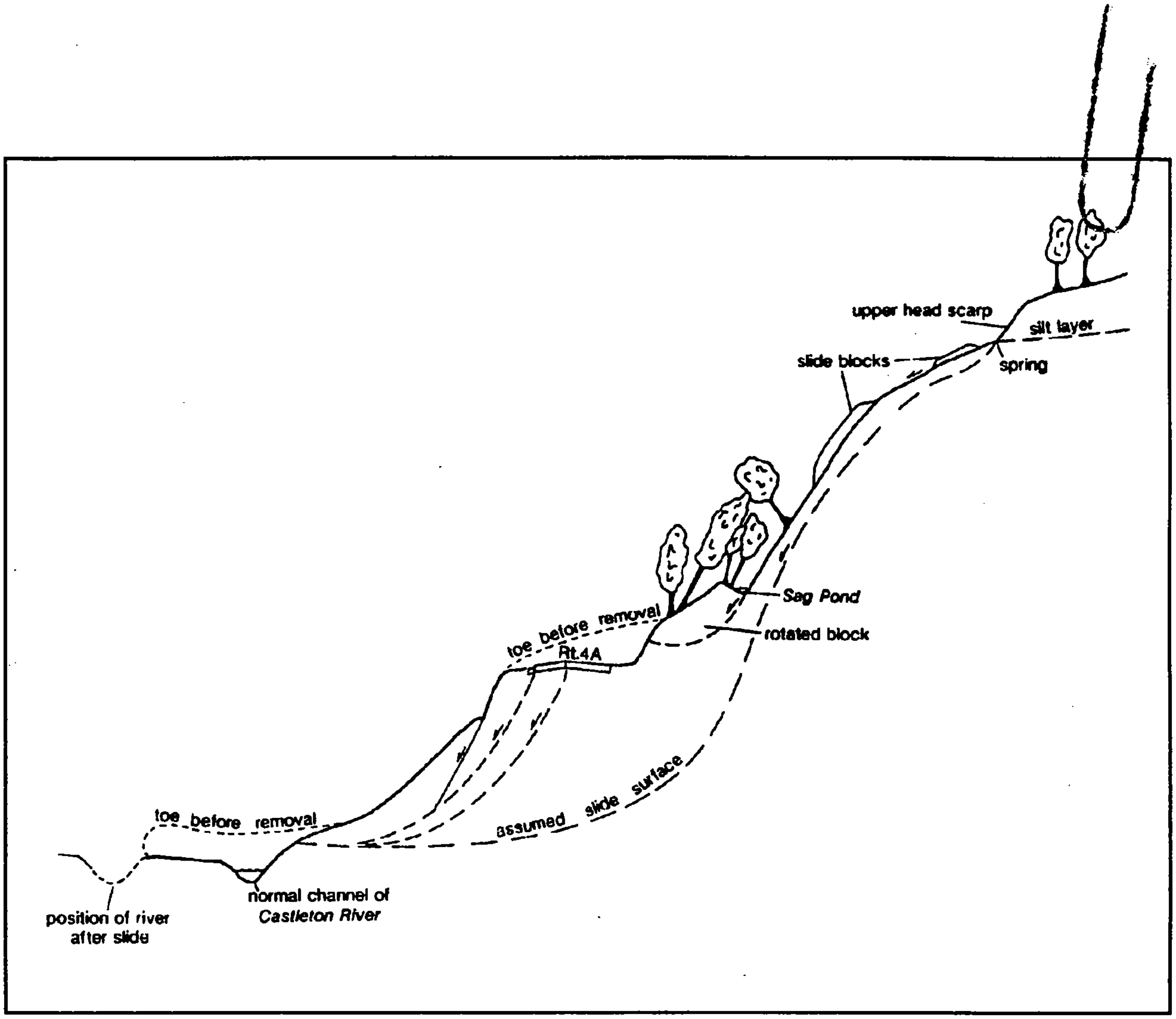

Figure 2A: Schematic of the Route 4A Castleton slide showing the assumed slide surfaces based on observed alterations to the surface topography.

Route 4A-Castleton River slide

This is an example of a landslide involving a basal till which occurred on Vermont Scenic Rte 4A in the Castleton River Valley near the village of Castleton. It was active for several years during the early to mid 1980's.

The soil materials in this slide includes a fine to coarse gray to gray-brown gravel with cobbles (GW) in a clayey silt to silty clay matrix. There is a trace to some fine sand (CL-ML) in the matrix (see ASTM, 1989 for soil group symbol definitions). This slide is complex. The lower part consists of a rotational slump mass and at least one translational slide block. The upper portion contains several translational slide blocks (fig. 2A).

Simply stated, there are at least two, if not more, individual translational slip surfaces in addition to the rotated mass. Two scarps are located on the slope; and open extension cracks have developed along the approximate center of the pavement, showing vertical displacement. The north side of the road has dropped somewhat below the south side (fig. 3).

The length from the top of the upper headscarp to the edge of the pavement measures 51 m. The width of the slide area above the route 4A pavement is 31 m. The overall width of this slide measures about 418 m east-west parallel with the pavement cracks. The gross vertical displacement is 10 cm when the incremental vertical displacements of the longitudinal pavement cracks are added.

The upper headscarp, composed of a gray till with a silty-clay matrix, has a vertical displacement of 1 meter. This material is normally very hard and impermeable except after periods of groundwater infiltration associated

Figure 2B: Sag pond and backward leaning trees in rotated block. Uphill is to the right.

with extensive rainfall. At a depth of a few centimeters below the base of this till-defended scarp, numerous springs issue steady streams of water. Samples taken from this area for laboratory examination indicated a moisture content of 12.8 percent.

The area, about half-way down slope, between the backward rotated mass and its scarp, contained sag ponds along with backward rotated trees (figs. 2B). The overall slope angle measured 26°, and the slope above the slide's uppermost scarp 14° to 15°.

Short (10.15 cm), 7.25 cm diameter, thin-walled Shelby tube samples were taken to obtain laboratory compressive strengths (fig. 4). Shelby sample VT8 failed at an axial load of 0.2150 kg/cm^2. The fracture that occurred at failure made an angle of 60° with the axis of the sample. Ten percent of the material in this sample consisted of pebbles up to 0.64 cm in diameter.

Sample VT9 did not fail with brittle fractures as did VT8 but bulged out plastically. The initial sample height was 6.35 cm, reaching 5.08 cm or a 20 percent reduction of its Initial height at failure. Besides being extremely soft, sample VT9 had a moderate amount of irregular void spaces, a massive aspect, and contained no coarse particles.

The two Shelby samples taken came from two different headscarps; they reinforce field observations which indicate a vague 'stratification' within the till. This phenomenon is quite likely due to more than one glacial advance (Stewart and MacClintock, 1969; Doll and others, 1970). The varying failure surfaces in this case may be related to differential strength and composition within the till and the resulting variations in pore-pressure.

Two bag samples of material from this landslide were analyzed with the X-Ray diffractometer, one untreated and the other glycolated, which helps enhance the untreated results. The diffractograms show that this soil is composed of Illite, Kaolinite, Muscovite, and Chlorite, in addition to silt and claysized quartz (fig. 5).

Using the classification of Varnes (1978) this slide is considered to be compound, both rotational and translational. The material can be classified as an engineering soil containing highly plastic clay ranging from wet to moist (Varnes, 1978). Moisture content was found to vary with the seasons and within different compositional units.

This slide is of the multistoried variety as defined by Ter-Stepanian and Goldstein (1969). The multistoried aspect is designated by the three stacked slide surfaces (fig. 2A).

North Branch and Great Brook, tributaries to the Winooski River

North Branch and Great Brook, tributaries to the Winooski River, have had impressive landsliding in the recent decade, causing much damage. In both valleys sections of highway were taken out (fig. 6A and B). These two locations are approximately 24 km apart.

North Branch and Great Brook landslides are of the

Figure 3: Vermont Route 4A looking east. North is toward road shoulder. Slide material has been cut back by the highway department. Expressions of the slide surfaces are the cracks in the pavement, one along the centerline, the other near the pavement edge to the left (see fig. 2a).

The North Branch, which flows through Montpelier, exits from the spillway of the Wrightsville flood control dam just north of the city. This dam was built in the 1930's to control the deadly floods in the Winooski Basin and the upper Connecticut River Valley (Baskerville and others, in press). The Wrightsville Dam Reservoir shows no signs of slope failures due to draw down of flood waters (see Baskerville and Ohlmacher, 1988).

Much of the reservoir perimeter is either rock-defended or has rock close to the surface. Downstream from the dam near the northern city limits of Montpelier the soils in the slide areas are mostly basal till. The North Branch landslide is of particular interest. An east-west reach of this stream coincidently parallels Gould Hill Road--also the north slope of the river (fig. 1).

The headscarp of this landslide runs a length of 112 m, cutting into the east-bound lane of Gould Hill Road for nearly half its width (fig. 6B). There are several en echelon longitudinal cracks running the length of the pavement parallel to and north of the headscarp. These have lateral and some downward displacement averaging

translational to shallow rotational types with shear strain along surfaces approximately parallel to the slope. This is due to variable pore water content (Varnes, 1978). Water infiltration related to antecedent rainfall helped create a pore-water pressure differential in the slope materials (Church and Miles, 1987; Neary and Swift, 1987). High velocity stream flow undercut the bank, removing slope toe support, triggering landsliding.

The soils in this reach of the Winooski River Basin consist of clean, medium-brown till along with dense basal tills, alluvium, and gray silty-clay rhythmites. Both North Branch and Great Brook flow wildly, assisted by steep gradients, during and after very intense rainfall of 80 mm and more. These factors are primary to the undercutting of their banks, triggering landslides.

The difference between the soils in the North Branch Valley and those in the Great Brook Valley is the lack of well-defined rhythmites in the former. The North Branch event undercut the bank, allowing the saturated slope to move down. This landslide removed part of the road on a side-hill cut in front of a house (fig. 6B). Landsliding also occurred farther upstream on meander bends.

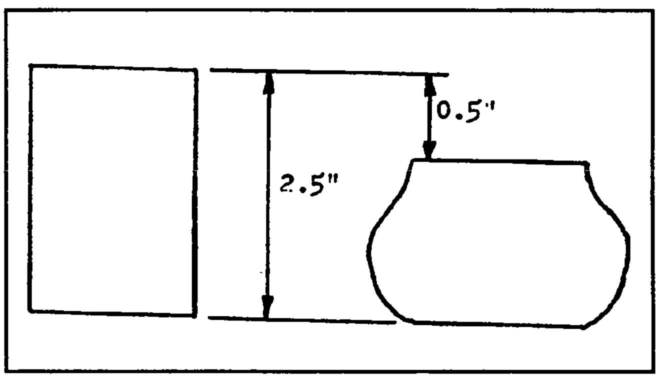

Figure 4A: Drawing of Shelby tube sample VT9 showing shape and measurements before and after undergoing uniaxial compression test.

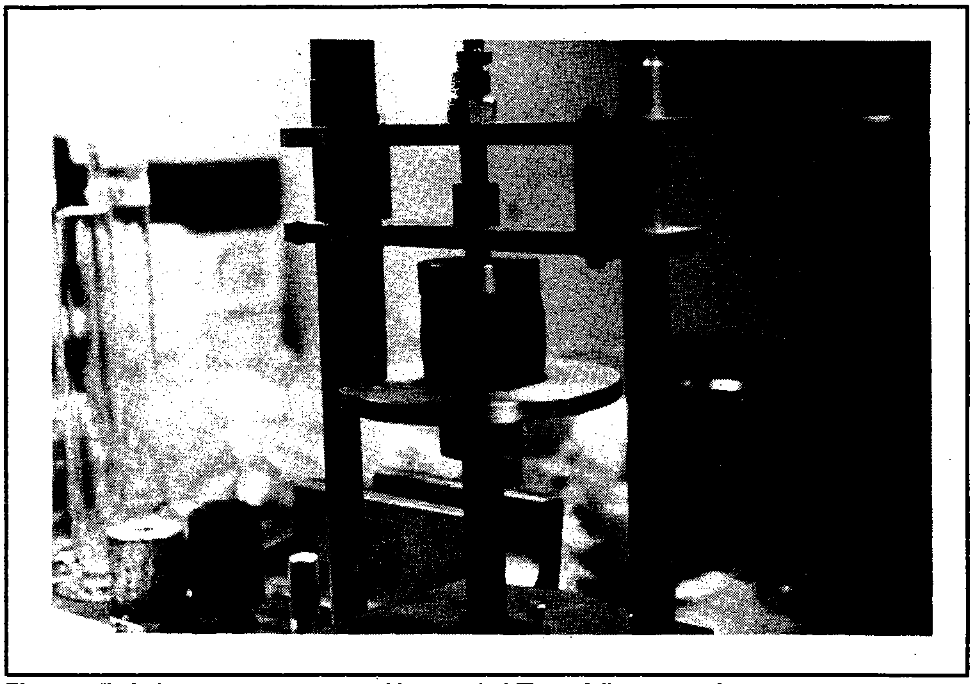

Figure 4B: Laboratory apparatus with sample VT8 at failure--note fracture.

10 cm. This provides additional shear surfaces that could move under the right conditions.

The Great Brook landslide headscarps are 100 meters long and more, with slopes averaging 60 m in height and slope angles of 30" and sometimes less. High up on the valley walls of Great Brook, slopes were still saturated on August 17, 1989 and exuding water after the heavy rains and floods of July. Water flow is controlled by sandy lenses (perched water) encased in very impermeable clayey basal till.

On Great Brook a $1,000,000 rip-rap bank protection had been built after a similar deluge and flood in 1984 destroyed many hectares of land, forest, and structures (fig. 7). The 1989 event removed the entire rip-rap bank, including sections of highway. This flood moved houses down stream (fig. 8). A half meter or more of sand and silt was deposited in much of the residential area on the south side of the village of Plainfield.

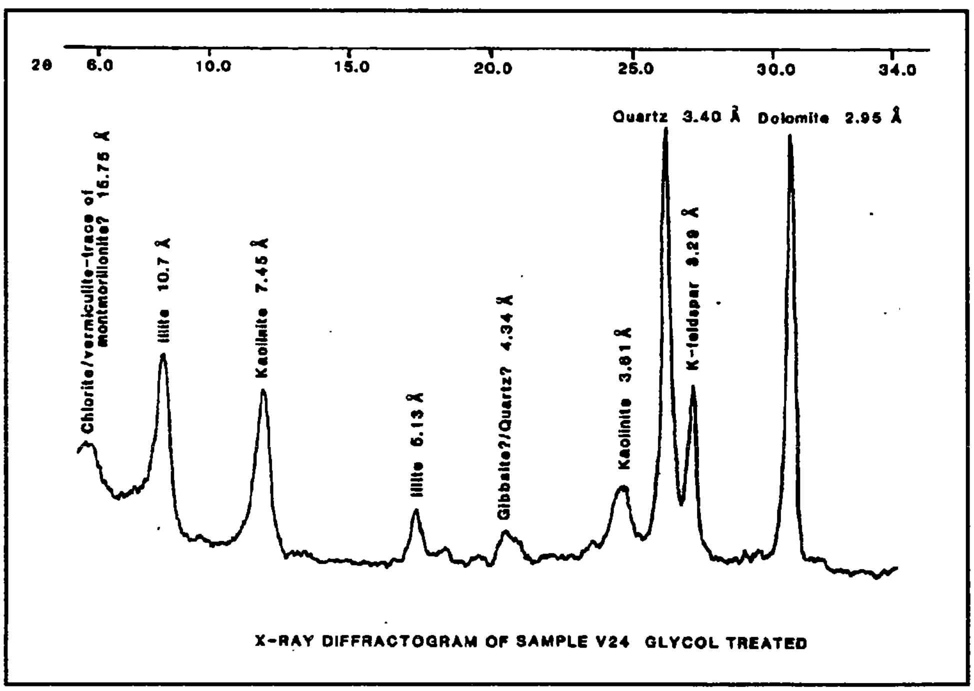

Figure 5: X-ray diffractogram obtained from -200 size material showing the presence of silt and clay sized quartz in addition to clay minerals.

Great Brook shifted its channel hundreds of meters and created new channels. Meanders were intensified and increased. The stream re-occupied parts of its floodplain that it probably had not used in a century or more.

Figure 6A: Road destruction in Great Brook Valley and landslide generated by undercutting.

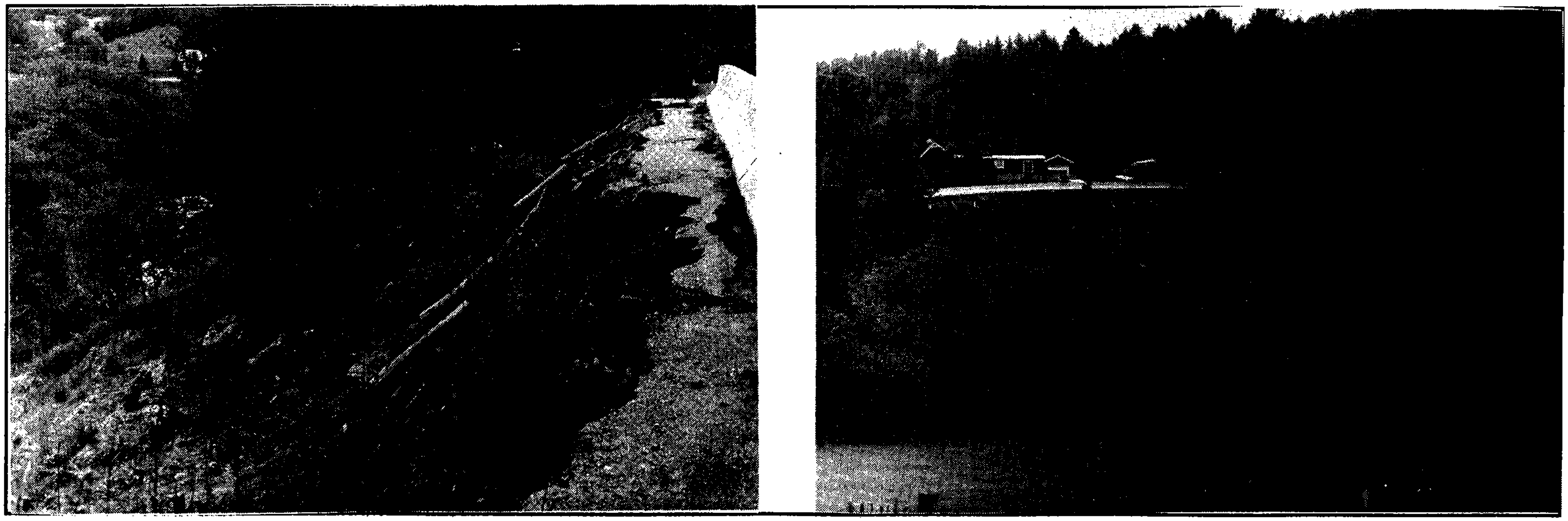

Figure 6B: Destruction of Gould Hill Road along North Branch. This is a tributary to the Winooski River. The site is at the north edge of Montpelier. **Left**–the headscarp of the landslide is along the centerline and on the south side of Gould Hill Road, which is defined by the Jersey barrier (white area, upper right). **Right**–looking north at the slide face. North Branch is just behind the tree-line to the rear of the open field. The house at the top is just behind the headscarp by a little more than half the road width.

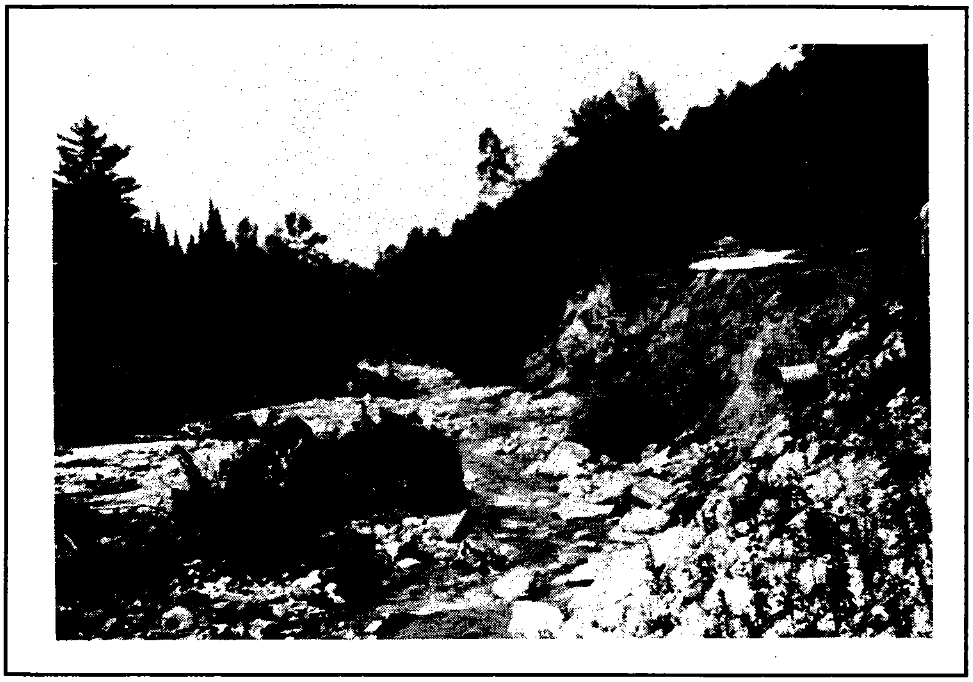

Figure 7: Looking downstream (north) along Great Brook, a portion of the highway and the rip-rap bank protection has been destroyed.

LANDSLIDES IN ROCK

The precursor for the northern New England rockfalls and rock avalanches was many seasons of frost wedging, hydrostatic pressure and vegetal growth in fractures. These elements place the rock masses at the brink of failure. Hydrostatic pressure from extended periods of precipitation, with or without snowmelt in spring, prepares the blocks for further movement (fig. 9). Either intensive rainfall or sometimes hurricanes can be the final trigger such as occurred in the Dorset Mountain slide (Ratte and Rhodes, 1977).

The Fairlee rock slide on I-91

A massive rock cut just off the southbound shoulder of I-91 between mile markers 85 and 100, [between Thetford and Fairlee (Fig. 1)], is known as the Fairlee Palisades. Bedrock in this area belong to the Ordovician Orfordville Formation, composed of the Post Pond and other volcanic units interbedded with schists and undifferentiated granitic rocks.

A major fault, the Amonoosuc, along with some minor faults, traverse the length of the landslide area. Several conjugate joint sets related to this faulting are present in these rock.

This rock mass faces in a southeasterly direction which makes it subject to persistent freeze-thaw during the severe winter months.

Figure 8: This house has been torn from its foundation and rolled over on its side by the Great Brook rain created torrent of 1989.

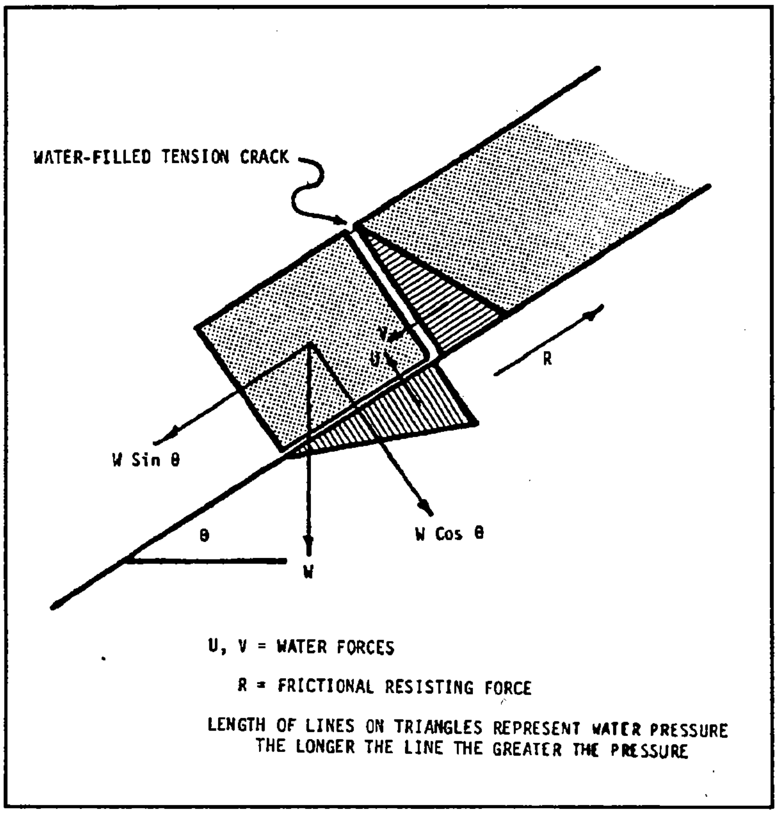

Figure 9: This schematic illustrates the effects of hydrostatic pressure on the movement of joint blocks in rock (after Hoek and Bray, 1977).

The spring rains and melt water from ice and snow produce hydrostatic pressure on the joint faces. This presure results in massive rock falls and rockslides, some blocking the entire roadway (fig. 1OA).

The Fairlee Palisades area has sustained rockfalls three times during the decade 1980 to 1990: April 5, 1985, March 9, 1987, and March 19, 1990. This section of I-91 was completed in the early 1970's, a rock cut standup time of over a decade before the first failure. Therefore, the time gaps do not indicate a reliable recurrence frequency that could be used for forecasting, but as of now 2 to 3 years appears reasonable.

The 1990 rockslide appears overall to be a wedge failure, based on the geometry of the space from which the slide rock originated (fig. 1OB). However, this concept is hard to grasp when one looks at the pile of debris in the roadway (fig, 1OA).

The Dorset Mountain debris avalanches

Dorset Mountain sits at the north end of the Taconic Range and is part of the Taconic Mountain series of thrust sheets. The Cambrian Breeze Formation, with its dolomitic sandstone layers, caps Dorset Mountain (fig. 1; Doll and others, 1961). Much of this unit is a phyllitic schist. These schists contain close-spaced joints (5 to 30 cm), and their foliation surfaces present additional planes of weakness. The dolomitic sandstone layers average about a meter in thickness and are highly jointed.

Sixty-five joint measurements were taken along the slide chute. They ranged from 4° to 75° NE or NW with dips from 72° to 89° NW or SE. Slope gradient at the upper levels of the mountain varies between 49% and 65%. Large volumes of water have been observed issuing from these fractures and foliation planes. This mountain is the headwater source for the Mettawee River.

The bedrock and soil on Dorset Mountain became saturated after sustained rainfall in early August, 1976. The immediate trigger was an intense storm on August 10, 1976. This storm initiated rock and soil slides at the 1159 m level in two chutes almost simultaneously (fig. 11).

The 1976 debris avalanches had at least a meter and more depth of fractured and decomposed rock available to feed the flow. The weathered schist becomes soft and punky with a dark rusty brown color. As the angular fragments of bedrock moved down slope, they became intermixed with the glacial till overburden.

In their course of travel, these debris avalanches were estimated to have incorporated at least 40 ha of trees. Several log jams remain at various elevations along the chutes. Other obstacles in the path of the debris avalanche, a hunter's cabin for example, were demolished.

Coarse material deposited by the slide shows an imbricated structure at the base of the deposits with fine material above. One of the 1976 slide chutes measured 38 m in width at the 774 m elevation. The main debris avalanche track, the final junction of the two chutes, stopped with a major log dam at elevation 472 m. This jam had to be cleared for fear that it would dam a lot of

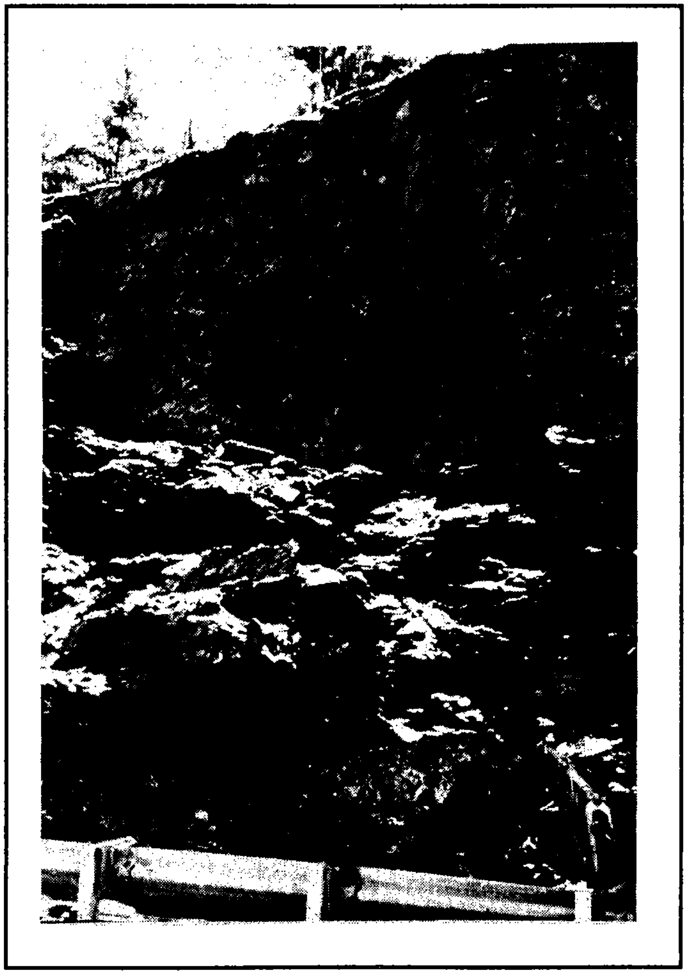

Figure 10A: Massive rockslide off the Fairlee Palisades, March 1990, blocking southbound Interstate 91.

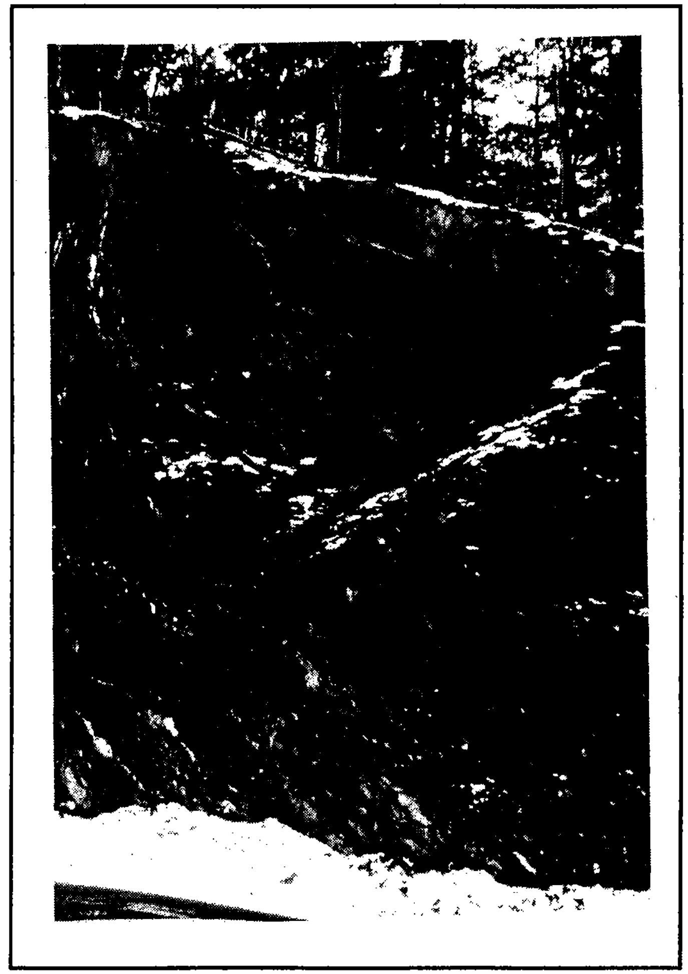

Figure 10B: Wedge-like area in rock face after removal of slide material.

Figure 11: Headscarps remain after the 1976 avalanches in south facing bowl of Dorset Mountain above Dorset Hollow. The distance is nearly one kilometer.

involved rock. The 1987 chute at the 774 m level measures 6 to 9 m across and is cut about 2 m below the floor of the 1976 chute.

At the 518 m level the Mettawee River had moved its channel eastward temporarily after the 1976 event. It cut down through the fresh-looking gray 1976 deposits and into older imbricated darkbrown clayey silt, sand, and gravel material. This channel is abandoned for the present. Both materials are from the same Source.

Penetrometer tests on the 1976 material produced results of <0.25 to 4.5 kg/cm^2 and Torvane readings of 2.5 to 4.0 kg/cm^2 and beyond the instruments range. The older material showed penetrometer results of 0.25 to 1.75 kg/cm^2 and Torvane 0.1 to 0.3 kg/cm^2.

water. This ponding would endanger homes farther down the valley in Dorset Hollow if the dam were to break.

As the energy dissipated against the log dam, water released from the flow carried gravel and cobble sized material downstream for another 1.6 km. This material completely filled in a reservoir behind a concrete weir (fig. 12). The total effective flow distance amounts to 3.7 km at an estimated velocity of 10 m/s.

The debris avalanche materials were highly fluidized--transported on a cushion of water and air (Ohlmacher and Baskerville, 1991; Varnes, 1978). This condition was surmised based upon the travel distance and the way material was deposited. For example, an 0.6 m boulder became lodged high in the crotch of a tree.

In the spring of 1987 (the exact date is unknown because the slide occurred high up on the mountain out of range of direct observation) a new slide occurred on one of the old 1976 chutes. This occurred after heavy spring rains and snowmelt. The depth of decomposition since the 1976 event was not enough to generate an adequate supply of material to move very far. Therefore, the slide toe only reached the 700 m level. There were a few saplings taken down, but otherwise most of the slide

On the assumption that the older material resulted from a pre-1976 debris avalanche, attempts were made to age date these deposits. Considering the known 1976 and 1987 events, Dorset Mountain slides should have a recurrence rate of about 10 years as a minimum. There was some carbonized material present near the upper contact of the pre-1976 material. The paleosol was not very thick and did not contain enough carbonized material for a C^{14} determination.

To obtain dates, tree ring cores obtained with an increment borer from trees downed in 1976 was examined. The trunks bored averaged 0.5 m in diameter. Core 1 indicated 126 years before 1976--1850, core 2, 140 years before 1976--1836, core 3, 87 years before 1976--1889, and core 4 181 years--1795. It is possible that the earlier trees (1795 and 1836) either survived landsliding for more than 100 years or the 1976 event was a 100-year event. This is suggested by the 1836 and 1889 tree ring dates. Considering that the 1987 event followed the same slide chute as the 1976 slide it is assumed that the older slide also used this route. Several old debris avalanche chutes were discovered around the Dorset

Figure 12: Reservoir in Dorset Hollow filled in by sand, gravel, and cobble material during the 1977 avalanche.

Mountain bowl. Also, large masses of rock at other places in the bowl can become topples which would trigger new slides or debris avalanches.

CONCLUSIONS

The few examples described detail the intensity and complexity of various types of landslides in mountainous terrain. This treatise points out some of the problems that engineers can face when designing or constructing highways or other projects in these terrains. Consideration should be given to problems that may arise in the adjacent hills and large rock exposures that could affect the project. These studies should be in addition to test borings for bridges, pavements and other pertinent structures along a highway right-of-way or any other type of structure in mountainous country.

Questions such as, "What effect will a major fault and its associated joint systems have on the stability of cut slopes through many seasons of freeze-thaw? What effect will clay layers in varves or rhythmites have on the stability of a soil slope when subjected to intense precipitation?", need answers. Detailed geologic mapping, soil and rock testing, along with a review of climatic conditions over past decades or more through the study of NOAA Climatological Data reports, may provide some of the answers and a key to safe design.

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LIST OF

HIGHWAY GEOLOGY SYMPOSIUM PROCEEDINGS THROUGH 1991

VOLUME I - First, Second, Third and Fourth Highway Geology Symposium

First Annual Symposium on "Geology As Applied To Highway Engineering"
April 14, 1950 - Department of Highways; Richmond, Virginia.

Parrott, W. T., "Synopsis of First Highway Geology Symposium."
*(Papers presented not published).

Second Annual Symposium on "Geology As Applied To Highway Engineering"
February 16, 1951 - Virginia Department of Highways; Richmond, Virginia

Cornthwaite, A. B., "The Adhesion of Bituminous Films To Highway Aggregates."
Cooper, B. N., Dr., "Geological Enterprise In Virginia - Present And Future."
Whitmore, Frank C., Jr., "The Importance of Geology In Military Highway Construction."
McConnell, Duncan, Dr., "Detrimental Minerals In Concrete Aggregate."
Granger, A. T.. "What Does The Engineer Expect of The Geologist?"
Hicks, L. D., "The Use of Plate Bearing Tests In The Thickness Design of Flexible Pavements."
DeBuchananne, George D., "The Control of Groundwater In Consolidated Rocks."
Woolf, D. 0., "The Identification of Rock Types."
Wheeler, Frank W., "The Construction of Highway Bridges And Separation Structures In Unconsolidated Sediments."

Third Annual Symposium on "Geology As Applied To Highway Engineering"
February 29, 1952 - Virginia Department of Highways and Virginia Military Institute; Lexington, Virginia.

Dobrovolny, Ernest, "Application of Geology To Highway Work As Practiced By The States, a Questionnaire Summary."
Nicol, Allen H., "Studies of Aggregates On Okinawa."
Nesbitt, Robert H., "Problem Minerals In Concrete Aggregates of The Southeastern States."
Anderson, Rex S., "Geology And Highway Engineering -A Continuing Mission."
Baker, R. F., "The Design of The Slopes of Highway Rock Excavations In West Virginia."
Hadley, Jarvis B., "The Geology of a Highway Slide At Gatlinburg, Tennessee."
Scroggie, Everette, "The Application of Geology To Bridge Foundations."
Parrott, W. T., "Geological Problems In The Design And Construction of Highways In Virginia."

Fourth Annual Symposium on "Geology As Applied To Highway Engineering"
February 20, 1953 - Morris Harvey College and West Virginia State Roac Commission; Charleston, West Virginia.

Gregg, L. E., and Havens, James H., "Applications of Geology To Highway Engineering In Kentucky."
Marshall, Harry E., "Some Experiences of The Department of Highways With Landslides In Ohio."
McNeal, John D., "The Application of Geology To Highway Subdrainage In Kansas."
Woods, K. B., Johnstone, J. C., and Yoder, E. J., "Some Engineering Problems Associated With The Preglacial Marrietta River Valley."
Price, Paul H., "The Landslide Problem."
Cavendish, Ray, "Landslides Affecting West Virginia Roads."
Eckel, Edwin B., "Contributions By The U. S. Geological Survey To Highway Engineering Research."
Philbrick, Shailer S., "Design of Deep Rock Cuts In The Conemaugh Formation."

VOLUME II - Fifth and Sixth Highway Geology Symposium

Fifth Annual Symposium on "Geology As Applied To Highway Engineering"
March 16, 1954 - The Ohio State University and the Ohio Department of Highways.

Melvin, John H., "Correlation of Geological Studies By Various Governmental Agencies In Ohio."
Hyland, John R., "Aids of The Ohio Geologic Survey For Highway Engineers."
Warrick, W. A.. "Application of Geology To Highway Engineering As Seen By a Highway Engineer."
Mason, Neil E., "Ohio's Experience In The Use of Geophysical Methods In Subsurface Exploration."
Norris, Stanley E., "Importance of Ground Water Studies To Highway Engineering."
Lewis, D. W., "Effect of Coarse Aggregate On Concrete Durability."
Mintzer, Olin., "Geology In The Engineering Curriculum, The Highway Engineer's Viewpoint."
Supp, Carl W. A., "Geological And Soils Engineering On Ohio Turnpike Project No. 1."
*Barger, Lewis, "Three Dimensional Aspects of Landslides."
*(Paper not published)

Sixth Annual Symposium on "Geology As Applied To Highway Engineering"
February 18, 1955 - Johns Hopkins University and The Maryland State Roads Commission.

Singewald, Joseph T., Dr., "The Role of The Geological Surveys In Geology As Applied To Highway Engineering."
Barber, Edward S., "Geology And Foundations."

Smith, Preston C., "Use Of Aerial Photographs In Engineering - Soil Mapping."

Melville, Phillip L., "Concrete Aggregate Reaction In Virginia."

Mobley, Arthur B., "Earth Resistivity Indicates Subsurface Geology."

Supp, Carl W. A., "Engineering Geology of The Chesapeake Bay Bridge."

Allen, Alice S., "Sources of Information On Ground Conditions."

VOLUME III - Seventh, Eighth, Ninth and Tenth Highway Geology Symposium

Seventh Annual Symposium on "Geology As Applied To Highway Engineering"
February 24, 1956 - N. C. State Highway and Public Works Commission; Raleigh, North Carolina.

Baker, Robert F., "Engineering And Landslides."

Mather, Bryant, "Application Of Petrographic Procedures To Highway Engineering."

Deuterman, Martin, "Investigation Of Bridge Foundations."

Laursen, Emmett M., "River Bed Scour At Bridge Foundations."

Laurence, Robert A., "Geologic Features Of The Eastern States As Related To Highway Engineering."

Bickel, J. 0., "Equipment Used In Geological Engineering."

McCullough, Charles R., "Aerial Photos And Highway Engineering."

Eighth Annual Symposium on "Geology As Applied To Highway Engineering"
February 15, 1957 The Pennsylvania State University and Commonwealth of Pennsylvania Department of Highways.

Eckel, Edwin B., "New Developments In The Study Of Landslides."

Gary, Carlyle, Dr., "Geological Information In Pennsylvania For The Highway Engineer."

Engel, Harry J., "Bridge Foundation Experiences."

Weeden, Harmer A., "Pedology Helps The Highway Engineer."

Bird, Paul H., "Experience In Designing Rock Slopes In New York State."

Baker, William 0., "Photogrammetry In Practice."

Ninth Annual Symposium on "Geology As Applied To Highway Engineering"
February 21, 1958 University of Virginia and Virginia Department of Highways.

Gay, Archer B., "Highway Geology And The Contractor."

Legget, Robert F., "Geology And Transportation Routes."

Maner, Alfred W., "Soils And Geological Engineering Partners, Not Competitors."

Rice, James M., "A Rapid Method For Determining The Resistance Of Ledge Rock To Freezing."

Dole, George J., "Ammonium Nitrate As An Explosive."

Tuttle, Curtis R., "Application Of Seismology To Highway Engineering Problems."

Tenth Annual Symposium on "Geology As Applied To Highway Engineering"
February 20, 1959 - Georgia Institute of Technology and Georgia Staie Highway Department; Atlanta, Georgia.

Felix, George D., "Geology As An Aid To Right-Of-Way."

Furcron, A. S.. Dr., "Distribution And Character Of Stone For Aggregate In Georgia."

Marshall, Harry E., "Design Considerations In The Treatment Of Soft Foundations."

Upham, Charles M., "The Use Of Geological Investigations In Foreign Consulting Work."

Seeger, Ralph W., "Highway Material Survey In West Virginia."

Fletcher, G. A., "Geology In Foundation Engineering."

Belcher, Donald J., "Applied Geomorphology."

Eleventh Annual Highway Geology Symposium
February 26, 1960 - Florida State University; Tallahassee, Florida.

Vernon, Robert 0., "The Geological Distribution Of Highway Base Course Material And Aggregate In Florida."

Mather, Bryant, "Petrology Of Concrete Aggregate."

Care., W. N., J., and Schmidt, W. J., "The AASHO Road Test: a Progress Report."

Michener, P. Z., "Preliminary Subsurface Investigation Of The Chesapeake Bay Crossing."

Fritz, Axel M., Jr.. "The MD Engineering Seismograph And Its Application To Highway Engineering."

Bruun, Per, "Beach Erosion And Protection In Florida."

Lynch, S. A., and Weaver, Paul, "Pavement Disruption By Recent Earth Movements."

Radzikowski, Henry A., "Rock And Earth In The Highway Program."

Twelfth Annual Symposium on "Geology As Applied To Highway Engineering"
February 10-11, 1961 - Engineering Experiment Station, Bulletin Number 24 - The University of Tennessee, Knoxville.

Young, Norman C., and Pierce, T. R., "Principal Highway Engineering Characteristics Of Some Tennessee Formations."

Goodwin, William A., "Evaluation Of Pavement Aggregates For Non-Skid Qualities."

Nichols, Donald R., and Yehle, Lynn A., "Highway Construction And Maintenance Problems In Permafrost Regions."

McNutt, Charles, "Highway Salvage Archaeology."

Officer, Charles B., "Use Of Continuous Seismic Profiler (Sparker) In Geologic Investigations For Vehicular Tunnel And Bridge Crossings."

Bailey, Reed W., "Madison River - Hebgen Lake Earthquake And Highway Problems."

Stuart, W. Harold, "Geological Conditions Complicating Highway And Railroad Relocations In The Northwest."

Moore, R. Woodard, "Observations On Subsurface Explorations Using Direct Procedures And Geophysical Techniques."

Dorman, C. W., "The Economics Of Natural Resource Valuation."

Gutschick, Kenneth A., "Altering Physio-Chemical Characteristics Of Clay-Bearing Soils With Lime."

Thirteenth Annual Highway Geology Symposium
March 16, 1962; Phoenix, Arizona.

Hall, Bruce M., "Status Of Geological Registration."
Gallaher, B. J., "Desert Materials - Types And Uses."
Eckel, E. B., "Use Of Engineering Geology Maps."
Smith, P. C., "Purpose Of The Highway Materials Inventory."
Sergent, B. D.. "Soil Mechanics In The Southwest."
Mitchell, Stanley N., "Problems Facing The Engineer And The Geologist In a Highway Department Employing An Engineering Geologist."
Kiersch, George A., "Regional - Aerial Geological Investigations In Highway Geology."

Fourteenth Annual Highway Geology Symposium
March 22, 1963 - A. & M. College of Texas; College Station, Texas, Texas Highway Department, and the Texas Bureau of Economic Geology.

Pelzner, Adrian, "Applications Of Agricultural Soil Surveys To Highway Geology."
Laughter, C. N., "Correlation Of Culvert Performance And Soil Conditions."
Henry, H. A., "Engineering Geology Operations In The Texas Highway Department."
Benson, Gordon R., "Geology - a Vital Part Of Subsurface Engineering In Illinois."
Welp, Theodore L., "Materials Geology, Co-ordinator Of The Aggregate Inventory."
Lemish, John, "Carbonate Aggregate Research."
Moore, Richard T., "The Effect Of Fractured Ground On Highway Structure Design."
Leith, C. J., and Gupton, C. P., "Some Geologic Factors In Highway Slope Failures In North Carolina."
Drew, E. D., "The Development And Utilization Of Engineering Geology In The California Division Of Highways."

Fifteenth Annual Highway Geology Symposium
March 19, 1964 - Missouri Geological Survey and Water Resources; Rolla, Missouri.

Helmer, R. A., "The Duties And Training of a Geologist In The Oklahoma State Highway Department."
Grimes, Walter W., "Geology And Foundation Problems of Glacial Drift In Eastern South Dakota."
Migliaccio, Ralph R., "Engineering And Construction Problems In The Valdez District, Alaska."
Landrum, J. D., "A Foundation Investigation of Cherokee Cave Under Route I-55, City Of St. Louis."
Lounsbury, W. R., and Schuster, R. L., "Petrology Applied To The Detection Of Deleterious Materials In Aggregates."
West, T. R., and Aughenbaugh, N. B., "The Role Of Aggregate Degradation In Highway Construction."
Heagler, John B., Jr., "Mineralogy And Soil Stabilization."

Sixteenth Annual Highway Geology Symposium
March 25-26, 1965 - University of Kentucky; Lexington, Kentucky.

Goodwin, W. A., "The Application Of Geology In The Benefication Of Aggregates."
Laughlin, G. R., Scott, J. W., and Havens, J. H., "Freeze-Thaw Characteristics Of Aggregates."
Lounsbury, R. W., and West, T. R., "Petrography Of Some Indiana Aggregates In Relation To Their Engineering Properties."
Sherwood, W. Cullen, Dr., "The Role Of Aggregate Type In Pavement Slipperiness."
Smith, Preston C.. "Landslide Research."
Struble, Richard A., "Shallow Subsurface Exploration Utilizing Airphoto Interpretation And Geophysical Techniques."
Dobrovolny, Ernest, "The Highway Research Board And Its Committee On Engineering Geology."

Seventeenth Annual Highway Geology Symposium
April 21-23, 1966 - Iowa State University; Ames, Iowa, Iowa State Highway Commission.

Burgat, Virgil A., "Engineering Geology In Kansas Highway Construction."
Mitchell, Robert E., "Seismic And Resistivity Matured."
Michael, Robert D., "Techniques Of Engineering Geology In Evaluation Of Rock Cores For Construction Materials Sources."
Blattert, Robert E., "The Effects Of Peat Deposits On Highway Design In Iowa."
Eversoll, Duane A., and Burchett, Ray, "Nebraska Geology And Highway Engineering Procedures."
Bruce, Richard L., "Landslides In The Pierre Shale Of South Dakota."
Kneller, William A., "A Study Of Chert Aggregate Reactivity Based On Observations Of Chert Morphologies Using Electron Optical Techniques."
Fritz, Axel M., "Engineering Geophysics: Its Use And Abuse."
Ledbetter, John F., "Aspects Of Using a Borehole Deflectometer To Diagnose An Unstable Rock Slope."
Welp, T. L. and Others, "Panel Discussion On Chemical And Physical Reactions Of Carbonate Aggregates In Concrete."
McElherne, T. E., "Introduction."
Lemish, John, "Background On Carbonate Aggregate Behavior In Concrete."
Mather, Katharine, "Test Methods Used In Investigating Alkali - Carbonate Reaction."
Gillott, J. E., "Concrete Performance As Related To The Behavior Of Carbonate Aggregates."
Newlon, Howard, "Chemical And Phvsical Reactions Of Carbonate Aggregates In Concrete."
Axon, Ely 0., "Concrete Performance As Related To The Behavior Of Carbonate Aggregates."
Mather, Katharine, "Waterways Experiment Station Experience With Alkali - Carbonate Reaction."
Newlon, Howard, "Physical Test Methods Used In Investigating Aggregates."
Dunn, James, "Distress Of Aggregate By Absorbed Water."
Lemish, John, "Concrete Weathering Studies."
Panelists and Audience, "Questions And Answers."

Eighteenth Annual Highway Geology Symposium
April 20-21, 1967 - Purdue University and Indiana Highway Commission; Lafayette, Indiana.

Woods, K. B., "Some Highway Problems Of The United States Correlated With Physiographic Provinces."
Miles, R. D., "Anyone Can Interpret Soil Groups From Aerial Photographs."
Struble, R. A., and Mintzer, 0. W., "Combined Investigation Techniques For Procuring Highway Design Data."

Berliant, R. F., and Sanborn, A. F., "Comprehensive Investigations Facilitate Design Of Interstate Highways Over Bottomland Soils."

Johnson, Robert B., "The Use And Abuse Of Geophysics In Highway Engineering."

Vineyard, Jerry D., and Williams, James H., "A Foundation Problem In Cavernous Dolomite Terrain."

Fredericksen, Walter, "Stabilization Of Abandoned Mine Under An Interstate Highway."

Hotler, C. F., "Soil Survey Practices In Indiana."

French, Robert R., and Carr, Donald C., "Geologic Factors Affecting The Exploration For Mineral Aggregates In The Indianapolis Area."

Malott, D. F., "Shallow Geophysical Exploration By The Michigan Department Of State Highways."

Panel Discussion: Preliminary Exploration For Highways With Emphasis On Local Problems."

Nineteenth Annual Highway Geology Symposium
May 16-17, 1968 - West Virginia Geological and Economic Survey, State Road Commission of West Virginia, West Virginia University Department of Geology, and West Virginia University College of Engineering; Morgantown, W VA.

Donaldson, Alan C., "Geology Of West Virginia With Special Reference To The Field Trip Area."

Hayes, Russel R., and Worrell, Donald T., "Geologic And Engineering Implications Of Poor Quality Sandstone From Simulated Highway Tests."

Leach, Richard C., " The Problem And Correction Of Landslides In West Virginia."

Thompson, Berke L., and Long, Donald C., "A Controlled Fill Over Sediments Of Ancient Lake Monongahela Near Fairmont, West Virginia."

Seeger, Ralph W., "Geological Investigations For a Trans-Andean Highway."

Smith, James D., "Geology Its Relation To The Design Of The East River Mountain Tunnels."

Cooper, Byron N., "Geology Of Big Walker Mountain Tunnel On Interstate Route 77, Wythe And Bland Counties, Virginia."

McGrain, Preston, and Dever, Garland R., Jr., "The Geometry Of Limestone Aggregate Sources In Kentucky's Appalachian Region."

Arkle, Thomas, Jr., and Brock, Samuel M., Jr., "The Geology Of Construction Sand And Gravel Resources In West Virginia."

Twentieth Annual Highway Geology Symposium
April 16-19. 1969 - University of Illinois; Urbana-Champaign, Il

Thornburn, T. H., and Liu, T. K., "Soils Of Illinois And Their Engineering Characteristics."

Brownfield, Robert L., "A Geotechnology Profile In Jo Davies County, Illinois."

Keene, Kenneth R., "Problems With Highway Cuts In Loess Near East St. Louis, Illinois."

Harvey, Richard D., "Fracture Surfaces Of Carbonate Aggregates: a Scanning Electron Microscope Study."

Gamble, James C., Hendron, A. J., Jr., and Way, Grover C., "Foundation Exploration For Interstate 280 Bridge Over Mississippi River Near Rock Island, Illinois."

Ferland, Jack A., "Seismic Mapping Of Cavities And Voids."

Thompson, M. R., "Properties Of Lime-Treated Soils."

Twenty-First Annual Highway Geology Symposium
April 23-24, 1970 - The University of Kansas; Lawrence, KA, State Highway Commission of Kansas, State Geological Survey of Kansas.

Wilson, Frank W., "Highway Problems And The Geology Of Kansas."

Goodfield, Alan G., "Rock Falls And Landslides Along State Highway 79 At Clarksville, Missouri."

Hammerquist, D. W. and Hoskins, Earl, "Correlation Of Expansive Soil Properties and Soil Moisture With Pavement Distress In Roadways In Western South Dakota."

Taylor, Charles L., "Geometric Analysis Of Rock Slopes."

Sennett, Robert B., "Geologic Factors In Design Of Excavated Rock Slopes."

Jenkins, Gomer, Jr., "Hydraulic Borrow Materials In Urban Areas."

Eversoll, Duane A., "Amphibious Drilling Rig."

Whitfield, John W., and Williams, James H., "Underground Transportation Routes And Quarry Practices In The Kansas City, Missouri-Kansas Area."

West, Terry R., "Application Of Remote Sensing To Highway Locations."

Stallard, Alvis H., "Remarks On Kansas Highway Research Project In Remote Sensing For Soils And Geologic Mapping."

Hartley, Alan, "The Influence Of Geological Factors Upon The Mechanical Properties Of Road Surfacing Aggregates."

Twenty-Second Annual Highway Geology Symposium
April 22-23, 1971 - The University of Oklahoma; Norman, OK, Oklahoma Geological Survey, Oklahoma Department of Highways.

Mankin, Charles J., and Johnson, Kenneth S., "Geology of Oklahoma -a Summary."

Hayes, Curtis J., "Engineering Classification of Highway - Geology Problems In Oklahoma."

Laguros, Joakim G., and Kumar, Subodh, "Predictability Of Shale Behavior."

Deere, Don U., and Gamble, James C., "Durability - Plasticity Classification of Shales And Indurated Clay."

Bayless, Glen, "Post-Construction Performance Of P-18 Bridge Abutments And Approach Fills, Eastern Oklahoma."

Parrott, William T., "Control of a Slide By Vertical Sand Drains, U. S. Route 220, Alleghany County, Virginia."

Merten, Fred K., "Straight Creek Tunnel Construction, Route I-70, Colorado."

Ivey, John B., "Highway Geology Feasibility Study, Luluabourg To Mbuji-Mayi Republic Of Congo (Kinshasa)."

Cleaves, Arthur B., "The Camino Marginal, Peru."

Gedney, David S., "A Design Approach To Rock Slope Stability."

Thompson, Berke L., "The Use Of Air As a Drilling Medium For Subsurface Investigation."

Johnson, B. J., "Penetrohammer, The Penetrometer Machine And Its Application."

Twenty-Third Annual Highway Geology Symposium
April 27-28, 1972 - Virginia Highways Research Council, Virginia Department of Highways, Virginia Division of Mineral Resources, Old Dominion University; Hampton, VA.

Jimenez, John, "Embankment Failures On The Tijuana - Ensenada Turnpike In The Lower California Peninsula, Mexico."

Branthoover, Gerald L., "Geological Engineering Investigation Of Talus Slopes In Lewiston Narrows, Pennsylvania."

Herbold, Keith, "Cut Slope Failure In Residual Soil."

Vogelsand, W. H., and Munyan, A. C., "Engineering Geology Of The Semi-Indurated Strata Of The Virginia Coastal Plain."

Thomas, Carl 0., "Remote Sensing Applications To Near Surface Geology."

Mitchell, D. A., and Brack, C., "Ecological Impact Of Hydraulic Construction Methods In Georgia."

Whitlow, B. S., "Investigation Of Deterioration In Concrete Roadway Slab Of The Robert E. Lee Bridge, Richmond, Virginia."

Baldwin, J. S., and Dawson, J. W., "Effects Of Angular Sands On Portland Cement Concrete."

Sennett, R. B., "Engineering Geology, Environmental Geology And Land Use."

<u>Twenty-Fourth Annual Highway Geology Symposium</u>
<u>August 9-10, 1973 - Wyoming State Highway Department; Sheridan, WY.</u>

Miller, Daniel, "Wyoming Geology" (Abstract Only).

Royster, David L., "Highway landslide problems associated with the escarpment areas of the Cumberland Plateau in Tennessee."

Wisehart, Richard M., and Wagner, J. Ross, "Slope stability and engineering consequences of a stream capture, Bluff Creek, northern California."

Edwards, Larry J., "Engineering Geologic Map Units For Highway Planning."

Patty, Tom S., "Accelerated Polish Test For Coarse Aggregate."

McKittrick, David P., and Gedney, David S., "Reinforced Earth For Highway Applications."

Bukovansky, Michael, "Engineering Geology And Rock Mechanics Help To Design a Freeway In Northern Spain."

Gilmore, John B., "Landslide Problems On The West Approach To Eisenhower (Straight Creek) Tunnel, Colorado."

Everitt, Martin C., and Holland, T. W., "The Shell Canyon, Wyoming Landslides."

Bauer, Edward, "Application Of Geology To Highway Construction In Mountain Terrain, Lovell-Burgess Junction, Wyoming."

Patty, Tom S., "Investigation Of Failing Concrete In Houston, Texas, Caused By Unsound Cement."

<u>Twenty-Fifth Annual Highway Geology Symposium</u>
<u>May 23-24, 1974 - North Carolina Department of Transportation & Highway Safety, N.C. Department of Natural & Economic Resources, and N. C. State University; Raleigh, NC.</u>

Keene, Kenneth R., "An Evaluation Of Sand Drain Installation Methods In Recent Alluvium."

Conrad, Stephen G., "Introduction Of The Geology And Mineral Resource: Of North Carolina."

Royster, David L., "Construction of a Reinforced Earth Fill Along I-40 In Tennessee."

Brown, L. F., Jr., and Fisher, W. L., "Environmental Atlas Of The Texas Coastal Zone And Its Role In Land Use Planning."

Sowers, George F., "Geological And Seismic Engineering For Nuclear Power Plants In The Southeast."

Welby, Charles W., "ERTS And Multispectral Photography."

Goughnour, Roger D., and Mattox, Robert M., "Subsurface Exploration State Of The Art."

Sams, Clay E., and Gardner, Charles H., "Engineering Geology Of I-26 Landslides, Polk County, North Carolina."

*Eades, James L., Dr., "Lime Stabilization." *<u>(Not published)</u>

<u>Twenty-Sixth Annual Highway Geology Symposium</u>
<u>August 13-15, 1975 - Idaho Transportation Department - Division of Highways; Boise, Idaho.</u>

*Bond, John G., "General Geology of Northern Idaho And Western Montana and its Implication on Highway Construction." *<u>(Paper not published)</u>

Mathis, Don H., "Application of Geotechnique To Design And Construction In North Idaho."

Larsen, Ronald E., "Seismic Designed Backslopes And Evaluation In a Structurally Disturbed Basalt Section."

Welch, J. David, "Environmental Geology Survey, Regional Land Use And Intermodal Transportation Planning, Northeastern Pennsylvania."

Steward, John E., "Use Of Woven Plastic Filter Cloth As a Replacement For Graded Rock Filters."

Garner, L. E., "Aggregate Resource Conservation In Urban Areas."

Patty, Tom S., "Petrography As Related To Potential Skid Resistance Of Paving Aggregates Used On Texas Highway Projects."

Bailey, Allen D., "Rock Types And Seismic Velocities Versus Rippability."

Johnson, Edgar L., "Traffic Safety Instrumentation, Foundation Landslide, Oregon."

Farnham, Paul R., "In-Situ Measurement Of Shear-Wave Velocities For Engineering Applications."

Bartholomew, Charles L. and Ireland, Herbert 0., "The Use Of Organic Topsoils As Construction Materials."

Kumar, S.; Annamalai, M.; and Laguros, J. G., "A New Analytical Approach To Split Tensile Strength Of Pavement Materials."

Thornton, Sam I., and Arulanandan, Kandiah, "Collapsible Soils, State-Of-The-Art."

Weatherby, David E., and Chapman, Ronald K., "Design And Installation Of An Earth Tieback Support System."

Denness, Bruce, and Smith, Robert M., "A Landslide In Pleistocene Deposits: Columbia (S. Am.)."

Wilmarth, Verle R.; Kaltenbach, John; Lenoir, William B., "Skylab Explores The Earth."

<u>Twenty-Seventh Annual Highway Geology Symposium</u>
<u>May 19-21, 1976 - Florida Department of Transportation and University of Florida; Orlando, Fl.</u>

Wisner, William A., "Florida Geology And Sinkholes."

Fountain, Lewis S., Sr., "Subsurface Cavity Detection: Field Evaluation Of Gravity, Radar, And Earth Resistivity Methods."

Morey, Rexford M., "Detection Of Subsurface Cavities By Ground Penetrating Radar."

Brooks, H. K., "Offshore Foundations Evaluation By Seismic Techniques"

Omes, Gildas, "Applications Of Geophysical Methods To The Detection Of Shallow Karstic Cavities."

Benson, Richard C., "Applications And Economics Of Engineering Geophysics."

Moore, Harry L., "Understanding The Phenomenon Of Piping."

McGrain, Preston, "Lapis-Type Features In Kentucky Karst Region."

Williams, Don E., "Underground Construction In The

Development Of Trails In a Large Cavern System In The Southern Ozark Mountains Karst Terrain."

Strohm, W. E., Jr., "Shale Deterioration Related To Highway Embankment Performance."

Kumar, Subodh, "Shrinkage Factor For Fill Construction Iowa."

Hammond, Tom, and Huckle, Horace F., "A Resume' of a study between the Soil Conservation Service and the National Aeronautics and Space Administration to determine if remote sensing could be technically and economicallv developed as a beneficial tool to assist soil scientists in selecting spots for field observations."

West, Terry R., "Evaluation Of Gravel Deposits Using Remote Sensing Data, Wabash River Valley North Of Terra Haute, Indiana."

<u>Twenty-Eighth Annual Highway Geology Symposium - August 10-12, 1977 - South Dakota School of Mines & Technology; Rapid City, South Dakota.</u>

Carrigan, Mark C., and Shaddrick, David, "Homestake's Grizzly Gulch Tailings Disposal Project."

Lovell, C. W., Bailey, M. J., and Wood, L. E., "Point Load Strength And Hardness Measures Of Shale."

Youell, James R., "Angle Hole Drilling Approach For Highway Engineering Data Collecting."

Burgat, Virgil A., "Cut Slope Design Based On Stability Characteristics."

Bauer, Edward J., "Reinforced Earth Fill On Steep Mountain Terrain Highway 14a, Big Horn County, Wyoming."

Hanna, Bruce E., "The Use Of Reinforced Earth Walls As Bridge Abutments."

Meadors, G. S., Jr., "The Geology And Construction Techniques Of The Second Hampton Roads Bridge Crossing, Norfolk, Virginia."

Grimes, Walter W., "Geotechnical Investigation For The Des Plaines River System Tunnels And Shafts."

Gardner, Charles H., and Tice, J. Allen, "A Study Of The Forest City Creep Slide On The Oahe Reservoir, South Dakota."

Glass, F. R., "Horizontal Drains As An Aid To Slope Stability On I-26, Polk County, North Carolina."

Rovster, David L., "Some Observations on the use of Horizontal Drains In The Correction And Prevention Of Landslides."

<u>Twenty-Ninth Annual Highway Geology Symposium May 3-5, 1978 - Maryland State Highway Administration & Geological Survey; Annapolis, Maryland.</u>

Cleaves, Emery T., "Geologic Contrasts Across The Fall Zone."

Rabchevsky, George A., and Brooks, David J., "Landsat Overview Of Fall Line Geology."

Langer, William, H., and Obermeier, Stephen F., "Relationship Of Landslides To Fractures In Potomac Group Deposits, Fairfax County, Virginia."

Winter, Ernest, and Beard, Brian, "The "0" Street Slide And Its Geologic Aspects, Washington, D. C."

Supp, Carl W. A., "Engineering Geology Of The Chesapeake Bay Bridges."

Hoover, Earl G., "Health And Environmental Assessment - a Major Geologic Concern On The Fall Line."

Frohlich, Reinhard K.; Maloney, James P.; and Lowry, Bruce E., "Control Of Vibrations From Commercial Blasting In Urban Areas."

Witort, Edward Anthony, Jr., "Effect Of Fall Line Geology On Design Of US-64, Rock Mount, North Carolina."

Guilbeau, Leonard H., "Design Alternatives For Construction Over Compressible Materials."

Monahan, Edward J., "Weight-Credit Foundation Construction Using Foam Plastic As Fill."

Lovell, Charles William, and Lovell, Janet Elaine, "Measure The Voids, Not The Solids."

Baker, Wallace Hayward, "Compaction Grouting Methods For Control Of Settlements Due To Solt-Ground Tunneling."

Brahma, Chandra S., "Significance Of Groundwater And Methods Of Ground Control In Tunneling."

Collison, Gary H., "Exploration And Geochemical Reporting For The Rock Tunnels And Station Of The Mondawmin Section Of The Baltimore Region Rapid Transit System."

Siegel, Ronald A.; Kovacs, William D.; Lovell, Charles William, "New Method Of Shear Surface Generation For Stability Analysis."

Brahma, Chandra S., and Ku, Chih-Cheng., "Geotechnical Perspective On Slurry Wall System."

<u>Thirtieth Annual Highway Geology Symposium August 8-10, 1979 - Portland, Oregon. *Sponsored by: Office of Fed. Hwy. Projects, Fed. Highway Administration.</u>

Chassie, Ronald G., "Landslide Tests Reinforced Earth Wall."

Mullarkey, J. Ray, "Fabrics In The Highway: The State Of The Art In Civil Engineering Applications."

Watkins, Reynolds K., "Structural Performance Of Buried Corrugated Polyethylene Tubing."

Hatheway, Allen W., "Revision Of The 1967 AASHTO Manual On Foundation Investigations."

Jackson, Newton, C., "Summary Of Use Of Sawdust For Highway Fills."

Ward, Timothy J., "Modeling Erosion And Sedimentation From Roadways."

Plum, Robert L., "Decision And Risk Analysis As A Practical Tool For Geotechnical Engineers And Geologists."

Kuenzli, James R., "Stabilization Of The Upper Portion Of the Hat Creek Landslide."

Royster, David L., "Landslide Remedial Measures."

West, Terry R., "Petrographic Examination Of Aggregates Used In Bituminous Overlays For Indiana Pavements As Related To Their Polishing Characteristics."

Miller, Henry J., "Geophysical Investigations Of Hampton Roads For Crossing Of Route I-664."

Pope, David H., "A Demonstration Project For Deicing Of Bridge Decks."

Lovell, C. W., "Compaction Prestress Makes A Difference."

<u>Thirty-First Annual Highway Geology Symposium August 13-15, 1980 Bureau of Economic Geology & The University of Texas at Austin in Cooperation with Texas Department of Highways and Public Transportation at Austin, Texas.</u>

Moore, H. L., "Karst Problems Along Tennessee Highways: An Overview."

Simpkins, W. W.; Gustavson, T. C.; Alhades, A. B.; Hoadley, A. D., "Impact Of Evaporite Dissolution And Collapse On Cultural Features In The Texas Panhandle And Eastern New Mexico."

Sansom, J. W., Jr., Shurbet, D. H., "Microearthquake Studies In Texas."

Abeyesekera, R. A., and Lovell, C. W., "Characterization Of Shales By Plasticity Limits, Point Load Strength And Slake Durability."
Wilson, John, "The Texas Natural Resources Information System."
Lo, T. Y. K., and Lovell, C. W., "The Geotechnical Data Bank."
Patty, T. S., "Engineering Petrography: Hwy Applications."
Allen, P. M., "Evaluation Of Channel Stream Bank Erosion In Urbanizing Watersheds In The Blackland Prairie, North-Central Texas."
Yelderman, J. C., Jr., "The Type Area Concept: a Practical Method Of Integrating Natural Resources With Planning, Development, Maintenance, And Landscaping Of Transportation Systems."
Whittecar, G. R., and Simpkins, W. W., "Drumlins At Potential Sources Of Sand And Gravel In Glaciated Regions."

Thirty-Second Annual Highway Geology Symposium
May 6-8, 1981 -Tennessee Department of Transportation - Division of Soils and Geological Engineering; Gatlinburg, TN.

Glass, F. R., "Unstable Rock Slopes Along Interstate 40 Through Pigeon River Gorge, Haywood County, North Carolina."
Tice, J. Allan, "The Hartford Slide - a Case History."
Aycock, James H., "Construction Problems Involving Shale In a Geologically Complex Environment, State Route 32 - Appalachian Corridor "S", Grainger County, Tennessee."
Mathis, Henry, "Temporary Landslide Corrective Techniques Avert Catastrophe."
Wright, E. M., "Remedial Corrective Measures And State Of The Art For Rock Cut Slopes In Eastern Kentucky."
Watts, C. F., and West, T. R., "A System For Rapid Collection And Evaluation Of Geologic-Structure Data For Rock Slope Stability Analysis."
Hale, B. C., and Lovell, C. W., "Prediction Of Degradability For Compacted Shales."
Wilson, Charles; Pope, David, and Sherman, William, "A Review Of The Progress Of The Wyoming Heat Pipe Program."
Jones, Don H.; Bruce S., Bell; and Hansen, Jack H., "The Application Of Induced Polarization In Highway Planning, Location, And Design."
Byerly, Don W., and Middleton, Lloyd M., "Evaluation Of The Acid Drainage Potential Of Certain Pre-Cambrian Rocks In The Blue Ridge Province."
Winchester, P. W., Jr., "Some Geotechnical Aspects -- Early Planning Along Corridor K, Appalachian Development Highway; Section Between Andrews And Almond, North Carolina."

Thirty-Third Annual Highway Geology Symposium
September 15-17, 1982 Colorado Geological Survey - USDA Forest Service, FHWA, Colorado Department of Highways; Vail, Colorado.

Bennett, Warren, "Experimental Compaction Of Collapsible Soils At Algodones, New Mexico."
Ivey, John B., and Hanson, Jerome B., "Engineering Geology, Relocation Of State Highway 91, Climax Mine Area, Summit County, Colorado."
Holmquist, Darrel V., "Slope Stability Consideration Of The Colorado State Highway 91 Relocation."
Pakalnis, Rimas, and Lutman, T., "Application Of Vacuum Horizontal Drainage."
Wyllie, Duncan C., and Wood, David F., "Stabilization Of Toppling Rock Slope Failures."
Robinson, Charles S., and Cochran, Dale M., "Engineering Geology Of Vail Pass I-70."
Hynes, Jeffrey L., "Geology Of The Glenwood Canyon Along I-70."
Bell, J. R.; Barrett, R. K.; and Ruckman, A. C., "Geotextile Earth Reinforced Retaining Wall Tesis."
Pell, Kynric, and Nydahl, John, "Geothermal Heating Of The Bridges And Tunnels In Glenwood Canyon."
Liang, Y., and Lovell, C. W., "Predicting The Strength Of Field Compacted Soil From Laboratory Tests."
Teme, S. C., and West, T. R., "Determination Of Friction Angle Values For Rock Discontinuities In Regard To Stability Of Highway Cuts."
Turner, A. Keith, "Computer Generated Maps."
Benson, Richard C., "Evaluation Of Differential Settlement Of Collapse Potential."
Sherman, William F., "Geotechnical Applications In Maintenance And Reconstruction Of The Existing Highway System."
Thornton, Sam I., "Fly Ash Leachate In Highways."

Thirty-Fourth Annual Highway Geology Symposium
May 2-4, 1983 - Georgia Department of Transportation, Georgia Geological Survey, FHWA; Atlanta, Georgia.

Dickerson, Robert T., "Investigation, Evaluation, And Quality Control Of Aggregate Sources In Georgia."
Bailey, Warren F., "Georgia Stabilized Embankment Wall Construction."
West, T. R., and Fein, M. R., "Geologic And Economic Aspects Regarding The Development Of An Underground Limestone Mine, Indianapolis, Indiana."
Leary, Robert M., and Klinedinst, Gary L., "Retaining Wall Alternates."
Nicholson, Peter J., "Innovations In Anchored Retaining Walls."
Abramson, Lee W., "Geotechnical Instrumentation Of Modern Retaining Wall Designs In An Urban Setting."
Barksdale, Richard D., and Dobson, Tom, "Improvement Of Marginal Urban Sites Using Stone Columns And Rigid Concrete Columns."
Trettel, Charles W., "Blasting Vibrations In An Urban Environment." Lambrechts, James R., "Southwest Corridor Project, Boston, Massachusetts."
Gruen, H. A., and Lovell, C. W., "Preloading Peat For Foundation Use."
King, John W., "Measurement Of Construction Influences On Adjacent Structures."
Sharma, Sunil, and Lovell, C. W., "Strengths And Weaknesses Of Slope Stability."
Collison, Gary H., "Geotechnical Data Collection For Design Of The Cumberland Gap Pilot Bore."

Thirty-Fifth Annual Highway Geology Symposium
August 15-17, 1984 - California Department of Transportation, and the Department of Geology, San Jose State University; San Jose, CA

Williams, John W., "Geotechnical Setting Of The San Jose Area, California."
Sparrowe, Thomas A.; Vassil, Vasiliki B.; and Young, Douglas T., "1984 Inventory Of Foothill Landslides, Santa Clara County, California."
Cotton, William R., "Engineering Geology Of The Carmel

Valley Road Rockslide, Monterey County, California."

Higgins, Jerry, "Characteristics Of Mudflows: Some Examples From The 1980 Mount St. Helens Eruptions."

Berkland, James 0.; Dahlin, Alan; and Remillard, Richard, "The Congress Springs Landslide Updated."

Durgin, Phillip, "Failure By Subsurface Stormflow In Melange Terrane."

Orr, William, "Correction Of Sycamore Draw Landslide, South of Big Sur, Monterey County, California."

Holzhausen, Gary R., "Slope Stability Monitoring In The Digital Age."

Alt, Jack, "Geologic And Seismic Considerations For Proposed Highway Bridge Sites Near Quito, Ecuador."

Griggs, Gary B., "Highway Protection And Maintenance At Waddell Bluffs, Santa Cruz County - Problems In An Active Geologic Setting."

Smith-Evernden, R. K., "Wave Erosion Of State Highway 1 Along The San Gregorio Fault Between Davenport And Pescadero, California."

West, Terry, "Detailed Office, Field, And Laboratory Analysis To Discern Rock Slope Stability, Interstate Highway 287, Northeastern New Jersey."

Chapman, K. Ronald, "Contracting For And Using Tiebacks For Landslide Stabilization."

Chen, Fred Y. M., "Geotechnical Design Parameters For Cut-And-Cover Stations And Tunnel Segments Of L. A. Metro Rail Project."

Schoeberlein, Elizabeth, and Slaff, Steven, "Overcoming Difficulties Encountered During Geotechnical Field Investigations Along Urban Transportation Corridors."

Sorensen, Mike, "Earthquake Ground Response Study For The Century Freeway, Los Angeles, California."

Hannon, Joe, and Walsh, Tom, "Final Results Of Embankment Performance At Dumbarton."

Thirty-Sixth Annual Highway Geology Symposium
May 13-15, 1985 -Indiana Department of Highways, Kentucky Transportation Cabinet, School of Civil Engineering - Purdue Universiiy; Clarksville, Indiana.

Gray, Henry H., "Outline Of The Geology Of The Louisville Region."

Mathis, Henry; Wright, Earl; and Wilson, Richard, "Subsidence of a Highway Embankment On Karst Terrain."

Moore, Harry, "The Pellissippi Parkway Extension - Geotechnical Engineering Karst Terrain."

Drumheller, Joe C., "Exploration And Repair Of Limestone Sinkholes By Impact Densification (abs)."

Amari, Dominick and Moore, Harry, "Sinkholes And Gabions: A Solution To The Solution Problem."

Killey, Myrna M. and Dumontelle, Paul B., "Illinois Landslide Inventory: A Tool For Geologists And Engineers."

Quinlan, James F., "Who Gets Sued When You Sink Or Swim, And Why: Liability For Sinkhole Development And Flooding That Affects Homes, Roads And Other Structures."

Anderson, Thomas C. and Munson, William E., "Tieback Walls Stabilize Two Kentucky Landslides."

Reeves, Ronald B. and Weatherby, David E., "Electrical Isolation Of Tieback Anchorages."

Richardson, David N., "Relative Durability of Shale - a Suggested Rating System."

Munson, William E., "Evaluation Of Geotechnical Designs For Shale Embankment Corrections."

Schuhmann, Mark J. and Schmitt, Nicholas G., "Use Of New Albany Shale For Subgrade And Pavement Stabilization."

West, T. R. and Hummeldorf, R. G., "Use Of Sonic Logs In Evaluating Roof-Rock Strength For An Underground Coal Mine."

Pfalzer, William, "Wick Drains."

Bleuer, N. K., "The Nature Of Some Glacial And Manmade Sedimentary Sequences And Their Downhole Logging By Natural Gamma Ray."

Lienhart, David A. and Stransky, Terry E., "Laboratory Testing As An Aid In The Design Of Cable Anchor Systems For Rock Reinforcement."

Nwabuokei, S. 0. and Lovell, C. W., "Predicting Settlements Within Compacted Embankments."

Bachus, Robert C., "The Effects Of Sample Disturbance On The Stress-Deformation Behavior Of Soft Sandstone."

Nieto, Alberto S. and Matthews, Peter K., "Moment-Driven Deformation In Rock Slopes."

Thirty-Seventh Annual Highway Geology Symposium on "Geotechnical Aspects of Construction in Mountainous Terrain"
August 20-22, 1986 - Montana Department of Highways and Montana Division - FHWA; Helena, Montana.

Berg, Richard S., "Geology of Montana".

Jones, Walter V., and Stilley, Alan, "Geotechnical Design Considerations For Road Construction Of An Active Talus Slope."

Torbett, C. Allen, and Ryan, Patrick T., "Statistical Analyses Of Factors Related To Rock Slope Stability In Eastern Tennessee."

Wright, Earl M., and Bukovansky, Michael, "Stability Problems Of Rock Cuts, US-23 In Eastern Kentucky."

Abramson, Lee W. and Daly, William F., "Analysis And Rehabilitation Of Aging Rock Slopes."

Miller, Stanley M., "A Time-Based Model To Help Evaluate Future Stability Of Cut Slopes."

Ciarla, Massimo, "Wire Netting For Rockfall Protection."

Watters, Robert J., Karwaki, Lyn, "Rockfall Mitigation As a Function Of Cost Benefit And Probability Assessment."

Wilde, Edith M., and Bartholomew, Mervin J., "Statewide Inventory And Hazard Assessment Of Deep Seated Landslides In Montana."

Moore, Harry L., "The Construction Of a Shot-In-Place Rock Buttress For Landslide Stabilization."

Turner, A. Keith, "Application of Personal Computer Models For The Stability Analysis Of Three Land Slides Near Vail, Colorado."

Thomaz, J. E. and Lovell, C. W., "General Method For Three Dimensional Slope Stability Featuring Random Generation Of Three Dimensional Surfaces."

Cowell, Michael J., Anderson, Ron and Anderson, Bob, "Polymer Geogrid Reinforced Soil Slopes Replace Retaining Walls."

Reeves, R. Bruce, "Design And Specification Of Tied Back Walls."

Aiyer, A. Kullathu, "Performance Of Internally Reinforced Soil Retaining System."

Franceski, John A., "Roadway Stabilization Using a Tieback Wall."

Thornton, Sam I., and Elliott, Robert P., "Resilient Modulus - What Is It?"

Thornton, Sam I., and Elliott, Robert. P., "Resilient Modulus - What Is It?"

Schulte, Michael P., "Dynamic Pile Monitoring And Pile Load Tests In Unconsolidated Sands And Gravels, Wyoming."

Olson, Larry D. and Church, Edward 0., "Survey Of Non-

Destructive Wave Propagation Testing Methods For The Construction Industry."

Norris, Gary, "Evaluation Of Nonlinear Stabilized Rotational Stiffness Of Pile Groups."

Ludowise, Harry, "Refraction Seismic Study To Explore a Borrow Source In a Remote Area."

Remboldt, Michael D., "Use Of Computer Spread Sheets In Geotechnical Design and Review."

Thirty-Eighth Annual Highway Geology Symposium on "Highway Construction In Unstable Topography"
May 11-13, 1987 - Pennsylvania Department of Transportation and Engineers' Society of Western Pennsylvania; Pittsburgh, PA.

Adams, William R., Jr., "An Empirical Model To Be Used In Evaluating The Potential For Landsliding In Allegheny County, Pennsylvania."

Brossard, Elizabeth A., and Long, Michael T., "Exploration And Analysis of a Proposed Roadway Over Organic Soils In Western Oregon."

Young, Brian T., and Shakoor, Abdul, "Stability Of Selected Road Cuts Along The Ohio River As Influenced Bv Valley Stress Relief Joints."

Leech, Thomas G.; Diviney, John G.; Janik, Charles T., "Landslide Stabilization In Hilly Urban Terrain."

Ackenheil, Alfred, "Ft. Pitt Tunnel North Portal Cut Slopes Revisited."

Watts, Chester F., and Frizzel, Earl, "A Preliminary Look At Simple Back Analysis Of Rock Slope Stabilities Utilizing Micro Computers."

Newman, F. Barry and Gower, T. R., "Geological Hazards Along Dorcon Road And LR 1094."

Bruce, Donald A., Dr., and Boley, Dennis L., "New Methods Of Highway Stabilization."

West, Terry R., "Highway Construction In The Lake Bed Deposits, Southwestern Indiana."

Hamel, James V., "Geological And Geomophological Investigation For Cultural Resource Evaluations."

Olson, Larry D., Church, Edward and Wright, Clifford C., "Nondestructive Testing And Evaluation Methods For Investigating The Condition Of Deep Foundations."

Bachus, Robert C., "Lessons Learned From European Practice On The Use Of Stone Columns For Site Improvements."

Diviney, John G. "Ground Modification Of Highway Embankment Foundation By Dynamic Compaction."

Hayden, Myron; Bloomburg, D.; Upchurch, S. B.; Williams, Ronald C., "Cone-Penetrometer Exploration Of Known Sinkholes."

Sheahan, James M., "Cut Slope Design For a Major Urban Highway In The Pittsburgh Area."

Moore, Harry L., "Karst vs. Highway Ditchlines In East Tennessee."

Thornton, Sam I., Kirkpatrick, W. E., "Cures For Slope Failures In Arkansas."

Leichner, Charles H., "Anchored Solutions For Unstable Topography."

Miller, S. M. Orbach; Canavan, William; and Kochel, R. Craig, "Assessment Of Landslide Potential Along Route 3, Southern Illinois."

Bonaparte, R.; Berg, R. and Butchko, S., "The Use Of Geosunthetics to Support Roadways Over Sinkhole Prone Areas."

Stokowski, Steven J., Jr., "Ground Magnetic Studies In Appalachian Valley Karst."

Voytko, Edward P.; Scovazzo, Vincent and Cope, Neil, "Rock Slope Modification Above The North Portal Of The Mt. Washington Tunnel."

Hazen, Glenn A., and Sargand, Shad, "The Effect On Highways Of Surface Subsidence Resulting From Longwall Coal Mining."

Wilshusen, J. Peter; Inners, Jon D.; Braun, Duane D., "Rock Slide On I-81, Northeastern Pennsylvania."

Markunas, Bernard, "Roadway Relocation Through Abandoned Municipal Dumps: A Case Study Near Hershey, Pennsylvania."

Thirty-Ninth Annual Highway Geology Symposium on "Construction To Minimize Environmental Impact"
August 17-19, 1988 - Brigham Young University, Utah Department Of Transportation, Utah Geological And Mineral Survey; Park City, Utah.

Doelling, Hellmut H., "A Brief Overview Of The Geology Of Utah."

Harty, Kimm M., "Geologic Hazards Of Utah."

Abramson, Lee W. and Hansmire, William H., "Geologic Engineering On The New Interstate H-3 In Hawaii."

Norrish, Norman I., and Lowell, Steve H., "Aesthetic And Safety Issues For Highway Slope Design."

Murtha, Geri Q.; Tiedemann, Robert B.; and Green, Richard W., "Construction Constraints - Wetlands, Runoff, Contamination."

Bruce, D. A., Dr., "Urban Engineering And The New Technologies."

Sotir, Robbin B., and Moore, William L., III. "Fill Slope Repair Using Soil Bioengineering Systems."

Curtin, Thomas J., and Tharp, Thomas M., "Stability Investigation Of Mt. Carmel Tunnel By Physical And Finite Models."

Rippere, K. H.; Williams, R. S.; and Funkhouser, M. R., "Investigation & Stabilization of a Developing Landslide At The Intersection Of US-89 And The D&RGW Tracks East Of Thistle, Utah."

Fan, J. C., and Lovell, C. W., "The Measured Slope Steepness Factor & Its Theoretical Analysis For Predicting Soil Erosion On Highway Slopes."

Verduin, J. R., and Lovell, C. W., "Reliability Analysis With PCSTABL5M."

Rana, G. M.; Smith, Jim; and Irani, Khodi, "Ground Water Influence On Highway Fill Slope Stability."

Moore, Harry L., "Oriented Pre-Split For Controlling Rock Slides."

Coffin, James L., "Installation Of An Underdrain System For Slope Stability."

Karably, K. B., and Humphries, R. W., "Talus Slope Stability Using Tie Back Anchors In Provo Canyon, Utah."

Leonard, Matthew; Plum, Robert L.; and Kilian, Al, "Considerations Affecting The Choice of Nailed Slopes As a Means Of Slope Stabilization."

Thommen, Robert A., "Steel Wire Rope Net Systems Used For Protection Against Rockfall & Debris Flow & All Other Purposes Of Protection."

Mitchell, David A., "4000 Bridge Foundation Investigations."

Capaul, W.: Wyllie, D.; Dunsmore, R.; Smith, R.; Winger, J.: Paroni, A.: Draeger, J., and Perfect, J., "Construction of a Tiei Back Soldier Pile Rock Retaining Wall Along I-90 In Northern Idaho, a Case History."

West, Terry R., "Construction of a New Interchange For The Indiana Toll Road, Complicated By Poor Soil Conditions And Presence

Of Sanitary Land Fill, Gary, Indiana."

Pihl, Roger, and Bowen, Tim, "Design And Construction Methodology For Rock Cuts In Glenwood Canyon."

Miller, Stanley M., "Modeling Shear Strength At Low Normal Stresses For Enhanced Rock Slope Engineering."

Burk, Robert L., and Moser, Kenneth R., "Spirit Lake Memorial Highway-Geologic Investigations In a Zone Of Natural Aesthetic Change."

Levine, Edward N.: Genson, Gordon, R.; Dye, Ronald R., and Slifer, James C., "From Geophysics To Design In An Environmentally Sensitive Area."

Scott, James H.; Burdick Richard G.; and Ludowise, Harry, "Interpretation Of Seismic Refraction Data On a Microcomputer."

Thornton, Sam I., and Elliott, Robert P., "Rapid Shear As An Evaluation For Base Course Material."

<u>Fortieth Annual Highway Geology Symposium</u> on "Symposium Venue"
<u>May 17-19, 1989 - Alabama Highway Department; Birmingham, AL.</u>

Bearden, Bennett L., "General Overview Of The Geology And Natural Resources Of Alabama."

Walker, Thomas H., "Engineering Geology And Geotechnical Engineering For a Preliminary Route Alignment Study For 25 Miles Of Arizona State Route 87."

Huang, Wei-Hsing, and Lovell, C. William, "Suitabijity Of Bottom Ash For Indiana Highway Construction."

Lockett, Larry, and Mattox, Robert M., "Geogrid Reinforcement For Cochrane Bridge Embankment."

Achilleos, E., and Lovell, C. William, "Update on STABL...PCSTABL5M."

West, Terry R., and Gordon, Quentin A., "Demolition And Removal Of Structures Prior To Land Reclamation."

Burns, Scott F.; Hadley, William 0.; Mutchler, Jack W.; Smith, Schaun M.; and Griffin, Paul M., Jr., "Slope Failures On Highway Embankments High In Shrink-Swell Clays: Prevention And Repair."

Nelson. K. Jeff, and Selvage, John R., "Design Applications Of The Welded Wire Wall."

Sharma, Sunil, "Integrated Slope Stability Analysis Using Microcomputers."

Morales, Carlo Hugo Rivera, "Honduras Highway Geology."

Wolosick, John R., "Contract Specification Options For Retaining Wall Design And Construction: Discussion Of Construction Alternates and Case Histories."

Sharma, Sunil, and Hardcastle, James H., "Finite Element Analysis of a Rib-Reinforced Steel Culvert."

Thommen, Robert A., Jr., "First Wire Rope Net Rockfall Protective Barrier Installed Ai The Grand Canyon National Park."

Wright, E. M., "Special Treatment Of Mine Openings In Rock Cut Slopes."

Humphries, Richard, and Sullivan, Randy, "Recent Highway Tunnel Projects In The Appalachian Mountains."

<u>Forty-First Annual Highway Geology Symposium</u>
<u>August 15-17, 1990 - New Mexico State Highway & Transportation Department, New Mexico State University Department of Civil, Agricultural & Geological Engineering; Albuquerque, New Mexico.</u>

Haneberg, William, "Geologic Hazards Of New Mexico."

Collins, Donley, and Swolfs, Henri, "Highway Damage Related to a Fault Near Pierre, South Dakota."

Barnes, Jamie, "Seismic Record Versus Geologic Record In The Southern Rio Grande Rift Region."

Moore, Harry, "Rockfall Mitigation Along I-40, Cocke And Cumberland Counties, Tennessee."

Watters, Robert, and Rehwoldt, Eric, "Slope Distress And Rock Fall Induced By The Presence Of Old Underground Excavations."

Duffy, John, and Smith, Duane, "Field Tests And Evaluation Of Rocknet Restraining Nets."

Elliott, Gordon, and Rippere, Kenneth, "Performance Analysis In Rockfall Simulation."

Rector, Edward, and Lueck, Richard, "Design Of Geogrid Wall With Wick Drains In Tucumcari, New Mexico."

Thornton, Sam, and McGuire, Michael, "Geogrid-Expansive Clay Embankment."

Cross, Richard, "Creating An Elevated Catchment Area Using a Precast Modular Wall System."

Deardorff, George, and Findley, David, "17 Miles To Mount St. Helens: Operational Aspects Of The Geotechnical Investigation."

Humphries, Richard; Elliott, Gordon; Cafarelli, Gerald; Hollenbaugh, John; and Geiger, Eugene, "Analysis And Design Of Tieback Wall No. 5 In Steubenville, Ohio."

Nicholson, Peter, and Johnston, Spark, "Alternative Methods For Retaining Walls."

Neel, Thomas, "T-Wall - Engineered For Economy."

Thornton, Sam, and Garret, Steven, "Slope Failure Probability For Layered Soils."

Chang, Chien Tan, "Federal Highway Administration's Technology Transfer Activities In Geotechnical Engineering."

Huddleson, Steven, "Data Acquisition System For Mechanical Dutch Cone Penetrometer."

Wisner, William, "Florida's Mineral Aggregate Control Program."

<u>Forty-Second Annual Highway Geology Symposium</u>
<u>May 28-31, 1991 - New York State Department of Transportation, NYS Geological Survey; Albany, New York.</u>

Bydlon, B., "Geotechnical Innovations on the Pennsylvania Turnpike"

Rudenko, D, Lawrence, W., and Ackermann, H.,"Seismic Refraction Technique Applied to Highway Design in a Strip Mine Area of Southwestern Pennsylvania"

Decker, M. & Jacobsen, G., "Evaluation of Acid Leachate Potential in Highway Construction"

Stokowski, S., "Quarry Layers - Stratigraphic Units that Serve the Public Interest"

Fischer, J. & Greene, R., "Roadways in Karst Terrane"

Mellett, J. & Maccarillo, B., "Highway Construction in Karst Terranes: Avoiding and Remediating Collapse Features"

Hardy, H.R., "Application of Non-Destructive Testing Techniques to Slope Stability and Sinkhole Monitoring"

Brandon, S., "Rock Slope Excavation and Stabilization Methods in Highway Construction: Interstate 287 Extension, New Jersey"

Hale, L. & Gansfuss, J., "Rock Slope Investigations at Selected Hudson Valley Sites"

Burke, J. & LeFevre, S., "Rock Slope Inventory, Evaluation and Remediation for Sections Along the New York State Thruway"

Cross, R., "Design for a Reusable Temporary Rock Catchment Barrier"

Bolton, C., "The Evolution of Rock Excavation and Stabilization in New York State: Emphasis on the West Point Quadrangle"

Abramson, L., "Geotechnical Exploration of Complex Tunnel Sites"

Dunn, J., Banino, G. & LeGrand, D., "High Quality Asphaltic Concrete Pavement Containing Chemically Treated Unsound Aggregate"

Hudec, P. & Achampong, F., "Improving Aggregate Quality by Chemical Treatment"

Parola, A. & Hagerty, D.J., "Highway Bridge Failure due to Foundation Scour and Instability"

Butch, G., "Measurement of Scour at Selected Bridges in New York"

Horne, W., Stevens, D. & Batson, G., "Ground Penetrating Radar Study of "Bridge Scour" In New York State"

McGuffey, V.C., "Clues to Landslide Identification and Investigation"

Thornton, S. & Garrett, S., "Slope Failure Probability for Mixed Layer Soils"

Baskerville, C. & Ohlmacher, G., "Northern New England Landslides"